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Paper No. 1131

A REVIEW OF CHICAGO PAVING PRACTICE.*

By P. E. Green, Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. W. W. Crosby, E. H. Thomes, Richard Lamb, William W. Marr, Nelson P. Lewis, and P. E. Green.

The matter of pavements in Chicago is in some respects a recent one, as compared with some eastern cities. Articles on New York 100 years ago state that that city was tolerably well paved; probably Philadelphia and Boston were also; certainly, Albany was, at the time of Fulton's first trip with his steamboat. Sixty years ago, however, Chicago had no pavements except plank and graveled roads, though in 1860 it was a city of 100 000 inhabitants. The condition of the streets can easily be imagined. There are photographs in existence showing that evidently every lot owner established his own grade for his sidewalk; and Clark St. had an appearance which can only be matched in the smallest towns of the West. In 1855 two or three squares were paved with limestone blocks, and at about the same time some cobblestone pavement was laid.

The first real pavement was the Nicolson block, a pine block similar to the present creosoted block, but untreated. This was laid on Fifth Ave., between Lake St. and South Water St., in 1856. The blocks were laid with 1-in. joints filled with gravel and coal-tar, the foundation being 2-in. plank on sand. The life of this

* Presented at the meeting of October 6th, 1909.

pavement was short, and it did not stay in favor very long, though a considerable yardage of it was laid before the Chicago fire in 1871. Up to 1865 about 25 miles of pavements had been laid, mostly pine block and gravel macadam.

After the fire, the grade of the whole down-town and near residence districts was raised by from 2 to 11 ft., the débris of the fire being used to make the filling. The writer, when making excavations for sewers and water pipes, during the last two or three years, has frequently encountered these old Nicolson blocks, still in a good state of preservation, after being buried in ashes, brick, etc., for nearly forty years.

It must be remembered that, until the grade was raised, a large part of the city was practically a swamp. In the Forties and Fifties the citizens frequently discussed the desirability of stone block pavements such as used in the East. It was the opinion then that it would not pay to spend much money on a pavement which was sure to sink out of sight in a year or two. It was this condition which brought about the use of plank roads. They were very serviceable for many years, and were active competitors of the steam railways of those days. The writer has read an article, written for one of the Chicago papers during the Fifties, which vigorously assailed the policy of abandoning the building of these roads, and went on to prove by quotation of freight rates that freight could be moved much more cheaply by team on such roads than by railway; that passengers could be transported just as rapidly and with less danger; and that, if the city were to spend on these plank roads the amount of money it proposed to give a certain railway, dividends of 30% could be earned.

In 1875 the city began to lay cylindrical cedar blocks, and for nearly twenty years this pavement was used more than any other. The blocks were 6 in. in depth, and, like the Nicolson blocks, were generally laid on a plank foundation, though a few streets had a macadam base. A great deal of this pavement still remains, but it is in very bad condition. When first laid, it had a nice, even surface, which was very attractive, and hundreds of miles of it were put down. About 1890 it began to fall into disfavor, and from that time its use declined, as its defects became glaringly apparent. Its first cost was one thing which made it popular, the average price per square yard being from $0.85 to $1.25.

In 1882 the first asphalt was laid, and during the same year the first granite blocks. The cost of these pavements was very high, and the use of asphalt spread slowly, but, from absolute necessity, the use of granite blocks spread very rapidly, and much of this pavement, laid from 1882 to 1886, is still in service. No care was taken to dress the blocks; they were only roughly squared, and were laid with wide joints. On account of these defects, the pavement was very noisy, but it was lasting. Brick was first laid about 1891. A brick pavement on Lake Ave., from 35th to 37th Sts., was laid in 1893, and is still in use, though badly worn. The bricks were not "repressed," but were vitrified. The foundation consisted of a course of brick laid flat. This pavement has given good satisfaction under heavy traffic.

One of the drawbacks to the rapid construction of good pavements in Chicago is the law under which such improvements are made, taken in connection with the financial condition of the city. Under the Illinois law, payment is made on the basis of the benefit derived by the adjoining property. In other words, whether or not a street has ever been paved, the adjoining property owners must pay for the entire improvement, unless the County Court, which confirms the assessments for the improvement, assesses a general benefit against the city. As the city has little money, and the bonding power is very narrowly limited, the policy is for the city to have the ordinance rescinded and the improvement abandoned if the assessment against it is more than it can pay, and so it works out that the adjoining property owners pay for at least 90% of all pavements laid in Chicago. The city does not pay for intersections. The various street railway companies pave their own right of way, a strip 16 ft. wide for double track. The whole arrangement is unjust, is a hardship on property owners, and should be changed. In an indirect way, it often injures property, as streets which should be repaved are allowed to remain for years, because property owners do not feel that they will be benefited by the improvement.

All work in Chicago which is to be paid for by direct assessment against abutting property owners is under the supervision of the Board of Local Improvements, composed of five citizens appointed by the Mayor. As the work done by this Board costs several million dollars every year, a considerable engineering force is required; but, as the finances of the city are in bad shape, rigid economy is necessary all

the time. Hence, under the present system, the engineering force is totally inadequate to do the work as it should be done. As in other public corporations, there is much duplication of work in the different departments of the Board of Local Improvements as now constituted. Undoubtedly, if this organization were to be remodeled, the number of men available in the Engineering Bureau would not have to be increased in order to do the work in a much more satisfactory way, but it is difficult to make changes in the method of doing public work. This statement is given as a reason for the methods which the engineers have adopted in order to do a great deal of work with a very small force. These methods are far from ideal, but the writer thinks that they obtain the best results, under the unfavorable circumstances.

In an ordinarily busy year, some three hundred streets and alleys are improved. As it is not possible for the small force of engineers assigned to this work to set stakes constantly for the curbing, grading, concreting, etc., much of this work must be done by the inspectors, who are not engineers. Line and grade are marked on permanent objects along the street, such as sidewalks, electric light poles, etc., and from these the inspectors and foremen must work. Ordinarily, some member of the engineering force visits each job every day, but he can rarely give more than an hour of his time to each piece of work, unless some particular point requires his attention.

In order that the inspectors and foremen may follow the engineer's instructions, each is furnished by the division engineer with what is called a "profile." This is not a profile, in the ordinary sense of the word, but is a plan of the work, with many explanatory notes. These notes show where the line is marked, the offset, the height of the grade marks from the top of the curb, the width of the roadway, the roadways of intersecting streets, the depth of the gutter below the curb at summits and inlets, the location of new catch-basins, etc. The inspectors and foremen are taught how to stake out the ordinary crown with Tees, after the curb is set. The curb, of course, is always put in before the concrete foundation, and hence is a guide to the men.

Of course, careless inspectors and foremen make mistakes occasionally, and there is trouble when the engineer makes his examination, but, as a whole, the system has worked better than could have been expected. Inlets and summits are sometimes reversed, crowns are too high or too flat, but nothing better can be expected until the

engineers in charge are given more men. At present an engineering force of about twenty men, with a salary list of about $33 000, handles $4 000 000 worth of work per year, scattered over the entire city. Fig. 1 is the "Profile" of a street paved by the writer in 1908. It will be noticed that this is not a detailed, accurate plan, but is really a memorandum for the guidance of the inspector and foreman.

Before discussing the different modern types of street pavements, various features, common to all in a greater or less degree, will be taken up. These may be classified under the headings: Width of Roadway; Crown or Form of Surface of the Roadway; and Drainage.

Width of Roadway.

The width of roadway is a question which causes the engineer more grief than almost any other. Property owners seem to have a general desire for a wide roadway. Many do not know, or more probably do not think, that the wider the roadway the more it will cost. Property owners, after receiving an assessment notice showing that the roadway will be reduced from the original width, have protested strongly, seeming to think that they were being defrauded of part of their rights, but when it was explained to them that their assessment would be much larger if the roadway were wider, they have been satisfied.

Chicago has a general ordinance for the width of roadways on streets of standard width. This, however, governs only the intersections of streets which have never been paved. When a street is to be paved, the ordinance ordering the improvement states the width of the roadway. The general ordinance specifies the following widths:

Width of street, in feet.	Width of sidewalks, in feet.	Width of roadway, in feet.
100	20	60
80	16	48
66	14	38
60	12	32
50	10	30
40	6	28
30	4	22

Probably any practicing engineer will agree that, for the average street, these widths are too great, and especially for very wide streets. Many residence streets have a width of 100 ft. between property lines,

and on such streets a 60-ft. pavement is the height of absurdity; it is expensive to lay and maintain, and certainly cannot vie in attractiveness with, say, a roadway of 38 ft., a grass space of 24 ft., and a sidewalk space of 6 ft., leaving 1 ft. between the walk and the property lines.

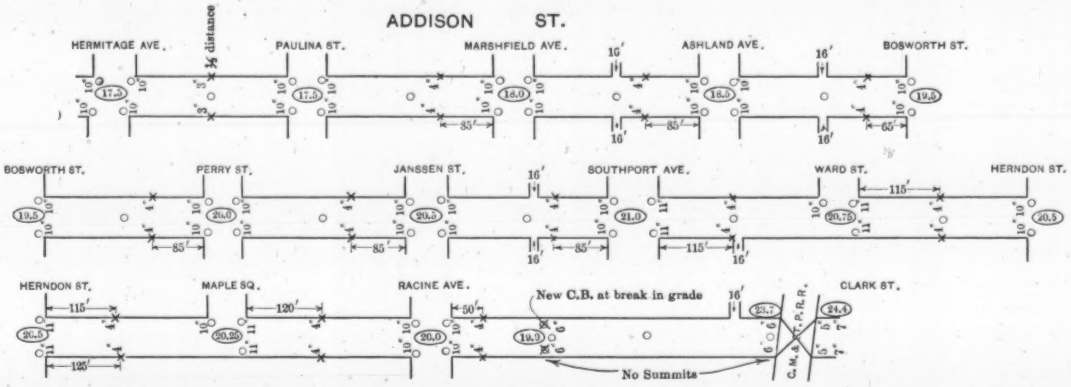
This tendency to make very wide roadways is more pronounced in small towns, which have not had many paved streets, than in the larger cities, which have had more experience. A certain town, having 3 000 inhabitants, proposed to pave its main street with a 60-ft. roadway, and then changed it to a 66-ft. roadway. An assessment for such a width nearly swamps the adjoining property owners. In another city, of 25 000 inhabitants, the main street has a paved roadway about 80 ft. wide; it looks like a deserted street on the busiest day, and actually gives the stranger a bad idea of the town.

If a street is wholly residential, and say 66 ft. wide, which is almost standard, a 30-ft. roadway is ample. This will allow two teams opposite each other to be backed up to the curb, and give a good passageway between, if the wagons are properly cranked. At street intersections, there is also ample room for vehicles of the fire department to swing around the corner. This is a vital point, and is sometimes overlooked by engineers, who go to the other extreme and advocate roadways which are too narrow. A long ladder-truck will have considerable trouble and delay in turning a corner on a narrow intersection, say 20 ft. There are generally electric light poles, lamp posts, etc., which may catch the ends of the ladders as they swing around. This objection, of course, can be largely overcome by building a wide sweeping intersection, but even that is not the only consideration. A very narrow roadway sometimes assumes too much of the appearance of an alley.

For an ordinary business street, not having car tracks, 38 ft. is a good width. A great many business streets in Chicago are of that width, and it is found to be ample.

If a street has a double line of car tracks, a wider roadway is necessary. The writer would then suggest a minimum width of 42 ft., but 48 ft. would be better. The reasons for this are obvious when it is considered that the ordinary street-railway right of way is 16 ft.

A congested district, such as "The Loop," in Chicago, furnishes a problem all by itself. In such a case the roadway can hardly be too



Profile of Addison St., from W. line of Clark St. to W. line of Hermitage Ave.

Roadway 44 ft. Grade marks 2 ft. High. Crown 7 and 10.

Line marked on south walk 13 ft. offset (from face of curb)

Intersecting Rdys. Racine Rdy. 38′ offset North 17.5′ South 14.0′ Maple Sq. Rdy.—offset See Engineer.
Herndon " " See Engineer Ward " 38′ " 14′
Southport " 48′ offset 16′ Janssen " 34′ " 16′.
Perry " 34 " 16′ Bosworth " 34′ " 16′.
Ashland " 38 " 14′ Marshfield " " See Engineer.
Hermitage " "See Engineer.

⊠ New Catch Basins.
○ Old Catch Basins.
× Summits.
⬭ Grades.

FIG. 1.

wide. But the sidewalk space is also congested, and must be taken into consideration. The roadways in "The Loop" are generally 48 ft. wide; part of State St. is 60 ft. wide. These are too narrow, but so are the sidewalks, and it is not possible to make much change.

Crown, or Form, of Surface of Roadway.

Apparently, there is little room for variation in the crown of the roadway. What is wanted is a shape which will drain rapidly and easily, and at the same time be as nearly flat as possible in order that it will not interfere with the freighting of heavy loads. For this purpose it is natural to use the circle or some similar curve. Many quite elaborate formulas have been suggested by various engineers as a solution of this problem. These generally take into consideration the width of the roadway, the grade of the street, and the desirability of having the maximum fall near the gutter. What most of them do not take into consideration is the fact that, in a flat territory like Chicago (and in some parts of nearly every city), it is absolutely necessary to form an artificial grade in the gutter. If the minimum depth below the top of the curb is 4 in., and the maximum is 10 in., it is obvious that a surface curve which depends entirely on the width of the roadway for its form, and, at the same time, establishes the center of the roadway at any fixed elevation, will not do. In the Chicago ordinances the form of the roadway is generally the arc of a circle. In construction work, however, it has been found by experience that this does not give the best results, and the parabola is almost always used. The formula is $y = \frac{b\,x^2}{a^2}$, in which b is the depth of the gutter below the grade of the center of the roadway, a is the half roadway, x is the horizontal distance from the center of the roadway, and y is the vertical distance below the grade.

This gives a crown which is not as flat at the center, and, at the same time, drops rapidly to the gutter in the last quarter; but, for most ordinary roadways, there is no practical difference between the two curves. It is also very simple to stake it out, and is easily taught to foremen, which is extremely important. The necessary stakes for the crown are generally set at the center and quarter points. In this curve the quarter points are always one-fourth of the total drop. This is very easily remembered, and the foremen are soon able to set the stakes accurately with wooden Tees. For very wide roadways

of, say, 100 ft., it causes too rapid a drop toward the gutter, and a curve intermediate between the parabola and a straight line must be used.

In regard to a crown of this form, however, it should be stated that it is intended for pavements having a rigid wearing surface which does not grind into dust. In other words, it does not apply to ordinary macadam, because it gives too flat a center, the wearing of the macadam soon forming a perfectly flat, or even a concave, surface which is impossible to drain. The writer considers that the nearer the section of a macadam roadway conforms to straight lines, dropping both ways from the center, the better the surface is likely to wear.

One of the important points to decide is how high the crown should be, or, in other words, how much the pavement should drop from the center to the gutter. It sometimes happens that a combination of circumstances, such as established curb grades and street railway tracks, necessitates a very low crown. The writer has known cases, where, on account of these conditions, combined with the requirements of drainage, the drop was less than 2 in. from the rails to the gutter. This was at a summit in the gutter. This is bad practice, however, and should be avoided when possible. For ordinary purposes for smooth pavements, on streets having a roadway of from 28 to 38 ft., 6 in. should be the minimum, and, for a macadam pavement, 8 in.; on wider streets, the drop must be greater.

The late Andrew Rosewater, M. Am. Soc. C. E., when City Engineer of Omaha, used, for the height of the crown, the following formula, which has met with a great deal of favor:

$$H = \frac{W(100-4P)}{5\,000},$$

in which W is the width of the roadway, and P is the percentage of grade.

This is a good rule for park roads or boulevards, or for any street having a stiff grade, when little crown is necessary, but it makes the crown too flat for level, or nearly level, streets, where drainage has to be secured by artificial grades, and where the streets are not constantly cleaned. In general, however, it is very good. It is practically the same as the 2% rule mentioned below.

One of the best general rules known to the writer is that used by the engineer of the West Park Board of Chicago, who makes the crown

2% of the roadway. This works very well on residence streets, but not at all well where there are street-railway tracks and the grade is nearly flat, as the tracks cannot follow the changes of the gutter grade, and these control the crown grade.

The Drainage.

The third general point to be considered—the drainage—is probably the most important. The whole life of practically every pavement depends largely on this. All pavements should be designed to take care of the drainage, and if any pavement is defective in that regard it will be a failure, even if it can stand the heaviest traffic. When the writer speaks of drainage, he does not mean entirely the sub-surface drainage, though that is very important. In a large city, as a general rule, there is not much trouble with the sub-drainage, because the city is thoroughly sewered, and the opening of cellars and other excavations has drained the surface and lowered the general ground-water level. Of course, a naturally sandy soil needs little drainage, but a clay soil must be watched. If it is normally in a wet condition, some provision for its drainage should be made, such as laying tile pipe across the roadway and connecting them to the sewer inlets. In this paper, however, the writer will consider only the surface drainage.

If the surface of the town is rolling, and naturally drains well, there will be little difficulty in keeping the pavements dry; but if it is very flat, like Chicago, drainage is an important problem. As has been intimated, drainage is effected by what may be called an artificial grade in the gutters. This is very readily secured if a combination concrete curb and gutter is built, in which case the gutters can be built first directly to the desired grade, and the pavement can then be built to follow it. But when a curb only is built, great care must be taken to conform to the grade desired, as the curb itself will give no indication of the proper place to locate a summit. The gutters should drain directly to a sewer inlet, and the inlets should not be more than 300 ft. apart—200 ft. would be better. If it is assumed that they are 200 ft. apart and the grade is level, the summit would be in the center, and a run of 100 ft. would be made each way. If the gutter is 4 in. below the top of the curb at the summit, and a drop of 6 in. in 100 ft. is desired for proper drainage, the inlet would be

10 in. below the top of the curb. This is as deep as desirable; if made deeper, people are likely to stumble over the curb, and the crown is made so stiff on the ordinary street that it is difficult to pull a heavily loaded wagon into the center of the street. It might be claimed that the drop of 6 in. in 100 ft. is more than necessary, but any practicing engineer who has had experience with city streets will agree that it is not too much. It must be remembered that in the ordinary residence street, which is generally not cleaned oftener than once in two or three weeks, but is flushed nearly every day, a large part of the street sweepings accumulates in the gutters. Added to this is the sidewalk dirt, which is also swept into the gutter. In park roads and in streets which are swept and flushed every day, the rate of fall may be reduced, and good results will be obtained with a fall of 3 in. in 100 ft. This will reduce the sewer inlets by one-half; but the street cleaning must be of the best.

Another point to be noted is the drainage of the intersections. Ordinarily, this is a source of great trouble, especially if the intersecting streets are not at right angles. In this case the result of the crown intersections is a large flat space. The greatest care must then be taken in staking the grade for the concrete or other foundation in order to make it conform as nearly as possible to the theoretical intersections of the curves. Wherever possible, it is the writer's custom to raise the center of the intersection an inch or two higher than the established grade, in order to facilitate the drainage. An ideal intersection would have three inlets at each corner, placing one back of each of the property lines and one at the curb-line intersection, and bringing the pavement cross-walks flush with the sidewalks at these corners. It is not very often that this arrangement can be followed, as it is somewhat more expensive than the ordinary way of having one inlet at the curb corner; but the result in keeping a dry intersection would soon pay for it.

There is one more general point which might be mentioned, as it applies more or less to all pavements, and that is the foundation under the wearing surface. In late years, the concrete foundation has come into use almost everywhere for streets having a heavy traffic. Its usual thickness is 6 in., which, as a general rule, is sufficient; for streets of very heavy travel 8 in. is better. In streets in which necessarily many openings have to be made, concrete is undoubtedly the

best foundation, but many residence streets and even business streets having a heavy traffic are but seldom disturbed.

Under such conditions, the writer is convinced that a thoroughly rolled stone base is the equal if not the superior of any concrete base. It keeps its line and grade just as well as concrete, if not better. As the heavy roller, used in compacting the stone, works continuously back and forth on this foundation for many hours, it will have every opportunity to discover any weakness in the subgrade. It may be said that the rolling of the subgrade should disclose this in any case, but, in addition to the subgrade rolling, the broken stone gets another tremendous pounding, to make a close, tight foundation, for which the concrete depends on its cement. Of course, a perfect concrete is inferior to no foundation, but, in ordinary work it is much easier to approach a perfect macadam foundation than a perfect concrete one. The writer believes that any engineer who has had experience in making either hand-mixed or machine-mixed concrete will agree with this.

In support of this general theory (which the writer is aware is opposed by many engineers), attention is called to the down-town streets of Chicago. Practically all are on a rolled foundation such as has been described, and, although the surface is badly worn, and the problem is complicated further by the poor construction of the car tracks, these streets are serviceable; and these pavements have been in service for from 15 to 25 years. The old granite blocks are rounded and worn, and will be replaced by a more modern surface soon; innumerable openings have been made in the streets, and this is an unfavorable location for such a foundation; but it is hard to believe that any concrete anywhere can show a better record. As an example of modern rolled streets, which show up very well, Harrison St., from Western to Rockwell, might be mentioned. This is an asphalt pavement on 6 in. of macadam, and was laid in 1903. It is in the best of condition, after 6 years of heavy traffic. Desplaines St., from Madison to Van Buren, of granite blocks on 6 in. of macadam, laid nine years ago, is a model surface to-day, though the traffic is very heavy. Wentworth Ave., from 55th to 63d Sts., was laid with brick on 6 in. of macadam, in 1897. Since then a conduit has been constructed in the street, and in 1908 the car tracks were reconstructed after being in bad condition for years, and yet the pavement

is serviceable to-day and is good for 5 years more. These are only a few examples. The only modification which would seem to the writer to be beneficial would be to put in an 8-in., instead of a 6-in., foundation on such streets.

In many places in which concrete has been used it would have been much cheaper to use a stone or gravel macadam. The writer knows of one small town in which the engineer provided a concrete base for a brick pavement, in spite of the fact that the street had been graveled for twenty years or more, and there was an abundance of good gravel in the vicinity.

In many cases a concrete foundation is a necessity, as, for example, in streets of very heavy traffic, streets having a bad foundation, such as wet clay, etc., and streets in which, undoubtedly, many openings will be made from time to time. As the concrete is generally from 6 to 8 in. thick, and is spread over a large surface, and not deposited in bulk, as in walls, abutments, etc., it is necessary to take more than ordinary care to obtain good concrete. A small sand pocket may make little difference in a large mass of concrete, but in a sheet like a pavement it may be the cause of a large hole which will ruin the wearing surface and extend with great rapidity under traffic. It is a vital necessity, therefore, to have inspectors at the delivery point and at the mixer, if one is used. If the concrete is mixed by hand, every care should be taken to see that it is thoroughly done, that too much water is not used in the mortar, causing the cement to be washed away, or too little, making a porous concrete.

The machine-mixing of concrete is one of the serious problems in street work. With many types of mixers, it is necessary, for a large part of the time, to have the mixer some distance from the work, such as at the next street intersection, etc. This means that each batch as mixed must be transported to the work in carts, wheel-barrows, etc. This is where the trouble comes in; the constituents of the concrete separate; in bottom-dumping carts, much of the "slush" or mortar, leaks out on the way, the heavier stone is shaken toward the bottom, and the net result is that the concrete is exceedingly uneven in quality when delivered. This trouble is largely eliminated by using a mixer placed close to the work and delivering the concrete on a board from which it can be shoveled into place. There are many places, however, in which it is very difficult, and sometimes impossible, to use such a

machine, and recourse must be had to hand-mixing or the distant machine. The writer would always prefer hand-mixing.

The proportions of the concrete are 1:3:6, using Portland cement, sand or limestone screenings, and broken stone, gravel, or crushed slag. The sand is standard torpedo. If limestone screenings are used, instead of sand, they must be clean, sharp, and free from dust or large flat pieces. There is no apparent difference in concretes made with sand or screenings.

One of the troubles in concrete foundations, caused by poor cement, poor mixing, etc., is the transverse cracks which appear later. These are especially bad for an asphaltic top, and cracks in this wearing surface are often caused by this fault and not by lack of bitumen, but longitudinal joints or cracks in the concrete appear to affect the asphaltic surface but little.

During the past two years, practically all the car tracks in Chicago have been reconstructed. This involved slight changes in grade in many cases, and often, on an asphalt street, the pavement was completely cut out half way back to the gutter. In laying the new concrete no particular effort was made to obtain a good bond with the old, other than by brooming a little and spraying it with water. It is probable that in most cases there is no real bond between the old and new concrete. After the concrete had set (generally in 4 or 5 days) the new top was laid. Some of this construction has now been down for 2 years, and no signs of cracking or failure have been seen.

The Wearing Surface.

In this paper no attempt is made to discuss macadam pavements of various types, as this subject has been treated very thoroughly in a recent paper by A. H. Blanchard, M. Am. Soc. C. E.* Strictly speaking, macadam is not suitable for a city pavement, but only for parks and light residence streets.

Brick Paving.—Ordinary vitrified brick is one of the most popular paving materials in small cities, but is not as popular in the larger ones. It requires a good foundation—as does any pavement—and the only problems in connection with it are the quality of the brick and the kind of filler. All block pavements require a sand cushion or bed of 1 or 2 in. under the blocks.

* *Transactions*, Am. Soc. C. E., Vol. LXI, p. 445.

In regard to the foundation, various substitutes, cheaper than a macadam or concrete base, have been used, but not with any great success. In some small towns having a sandy subgrade the bricks are laid directly on the sand after rolling the subgrade, but the pavement soon wears into ruts under any large amount of traffic. A base course of bricks instead of concrete has also been used as a foundation, but, if laid properly, this is as expensive as concrete, or more so, and is not as good.

The quality of the brick is determined by water absorption, specific gravity, and abrasion tests. To these might properly be added a test of the uniformity of shape. It happens frequently that bricks which are otherwise good have been warped so much by the firing that they have to be rejected on account of their shape. If a uniformity test were applied, and the provision were made that not more than a certain percentage of the bricks should depart from the required shape, the pavement would undoubtedly be improved. The form of this provision might be as follows:

"Not more than 5% of the bricks shall depart a greater distance than ¼ in. from the standard block furnished by the manufacturer. A greater percentage than this shall be cause for the rejection of the entire lot from which the samples tested were taken."

It is not possible to give exact sizes for paving brick, as those of different manufacturers vary slightly, but it can be specified that they shall not be smaller than a certain size, say 3 by 4 by 8 in., or larger than 3½ by 4 by 10 in., and this is generally done.

The water absorption test is very important, as it measures the porosity of the brick. If the weight of water absorbed is more than a very small percentage of the weight of the brick, the latter is almost sure to go to pieces in two or three years under the freezing and thawing action of winter. The Chicago specifications limit the quantity of water absorbed in 72 hours to 3% of the weight of the brick, and some of the engineers desire to reduce this to 2 per cent. This would probably be done if it were not for the fact that so many bricks are rejected under the 3% clause; for it is thought that at the 2% rate no brick at all could be obtained.

The specific gravity required is generally about 2.1. The various manufacturers have no trouble in meeting this specification, and it has been found by experience that this gives a good heavy brick.

The abrasion test is also very important. A brick may be dense and heavy, but so brittle that it will not stand pounding. For this test the rattler is used. It consists of a steel cylinder, about 28 in. in diameter and 20 in. long. The sides are composed of steel staves having open spaces between to permit the escape of broken fragments. The size of the charge is about 15% of the volume of the rattler. Inside the rattler a number of pieces of loose heavy castings are placed. The rattler makes 30 rev. per min., and is run for an hour; bricks which lose more than 20% of their weight are rejected.

Disregarding the quality of the brick, the point over which there is most controversy is probably the filler. Three fillers are in general use: sand, bitumen, and cement grout. Sand is the oldest and cheapest form, but is not now used to a great extent. The objections to it are that it does not make the joints water-tight, and that flushing the street forces the sand out of the joints and loosens the brick.

The most popular material is either a cement grout or a bituminous filler such as coal-tar, pitch, or asphalt. The writer believes that, as far as wear is concerned, there is little to choose between tar, asphalt, and cement grout fillers. In favor of the cement grout, it is urged that it makes the pavement approach a monolithic whole, protects the edges of the bricks, and is most easily cleaned and kept clean. Its chief objections are that it increases the noisiness of the pavement, and that after grouting, the pavement must be undisturbed for several days in order to set thoroughly. In any street of large traffic, it is nearly impossible to comply with this last requirement, and a corps of police would be needed to keep the traffic off the apparently finished street. For this reason alone, the authorities in Chicago have entirely abandoned grout, and now use a bituminous filler exclusively.

The grout is generally made in the proportions of 1 part of cement to 1 or 2 parts of sand. It is generally maintained by engineers that it must be carefully mixed in a suitable box, and that every precaution must be taken to prevent the separation of the sand and cement. From his experience, the writer believes that this idea is good, but its practical attainment is difficult and unsatisfactory in any form of box. Formerly, it was his custom to insist on the box, but there was so much trouble that one of the contractors was finally permitted to try his own scheme. He made a thorough mixture of the dry sand and cement and swept it into the joints, thus filling them about half

way to the top. A spray of water was then applied and the grout was formed in the joints. Plenty of water was used, care being taken that it was not forced directly into the joints so as to wash out the mixture.

For a second course, sand and cement were mixed dry in piles directly on the pavement, and then the hose was turned on it. By using squeegees and the hose, this was thoroughly mixed, and the whole was then spread over the pavement and into the joints. The results have been of the best. During the past two years, there has been occasion to examine several openings made in pavements grouted in this way, and there is yet to be found a bad joint. In one particular case, where the adjoining street was afterward paved with asphalt, and the brick pavement at its intersection was exposed and, for about half the width of the brick, undermined, the grout joint held it up in spite of the fact that the material wagons were constantly driving over this intersection.

The advantages of a bituminous filler are that it tends to make the pavement less noisy, and also makes any repair easier than when grout is used, for it is very difficult to make openings in the latter if it has been properly applied.

The bituminous filler is either a pitch, which is the result of the distillation of coal-tar; or some asphaltic compound. The name pitch simply means coal-tar from which the water and lighter oils have been driven off by distillation, the product being hard at ordinary temperatures. If the distillation is carried too far, parts of the heavy oils are also driven off, and the result is a hard, dense product, which cracks very easily, and is not suitable for the purpose.

The pitch filler is either poured on the pavement from buckets and squeegeed into the joints, or else poured directly into the joints with a special bucket. The latter method is a little more expensive, but is the better, as the pitch on top of the brick is a nuisance.

The asphaltic filler is probably better than the pitch filler, as it does not crack so much in cold weather, nor does it lose its oils as rapidly as the tar products. In other words, a filler having an asphaltic base is more stable than one having a coal-tar base; but, as a general rule, it is also about twice as expensive. It is always applied with a special pouring bucket, and is never squeegeed.

If a grout filler is used, some provision should be made for expansion. Generally speaking, an expansion joint of ½ in. along the gutter,

and ½ in. longitudinally every 50 ft., will be ample. If there are street-railway tracks, it is advisable to lay the bricks for about three courses longitudinally with the street, and use a bituminous filler, not on account of expansion, but because of the incessant vibration caused by the cars, which tends to break the bond of the grout.

Secretary Blair, of the National Paving Brick Manufacturers Association, recommends the total elimination of the transverse expansion joint in brick paving, maintaining that the longitudinal joint is sufficient, but the writer is not prepared to agree with his reasoning.

Each of the various companies promoting tar, asphalt, and cement grout fillers insists that its product is the best. During the past three years the writer has made openings in many brick pavements in which a pitch filler had been used, and has nearly always found the joints well filled, even when the pavement was many years old. The same could no doubt be said of the asphaltic filler, but it costs more than twice as much as the pitch, and it has not been proved that it will really give any better service, though from its nature it should be somewhat superior to pitch.

Although cement grout at first makes a smoother surface, in the course of three or four years the grout in the joints is broken by the blows of countless horse-shoes, and after that there is practically no difference between its appearance and that of a tar-filled street. The joints then rapidly wear down, and become wide and rough.

An example of exceedingly good service given by brick may be seen on La Salle St., from Randolph to Madison Sts. This pavement has been in service for 14 years, and it is still usable, though nearly worn out. The traffic on this street is enormous, and innumerable openings have been made.

The principal objections to a brick pavement are its noisiness on residence streets and the wear of the edges on heavy traffic streets. Its advantages are that it is easily cleaned, is not affected by the weather, and, under proper conditions, has a very long life.

Stone Block.—The best grade of pavement, considered purely from a wearing standpoint, is the stone block. It is made with dressed blocks, generally about 5 in. wide, 5 in. deep and from 8 to 10 in. long. The block should be a hard stone of medium grain, which will resist the weather and not wear smooth and round. Sandstone of various kinds, granite, and what might be called "near-granite," such

as quartzite, which is really a sandstone, rhyolite, etc., are used. Various kinds of artificial blocks, such as iron slag, etc., have also been tried, but, in this part of the world, at least, have not met with much favor.

Much better stone block pavements are now laid than formerly. This is due largely, if not entirely, to more careful dressing, which makes a smoother and more regular block; and more care is taken in laying them. Formerly in Chicago the blocks were only roughly dressed, and were laid with wide joints. The result was that the edges were very much rounded under traffic. No investigation was made as to whether or not the blocks became slippery under traffic. Blocks which in the past have worn badly will not now be accepted. Samples of the so-called granite block to be used are required by the authorities when a bid is made, and the location of the quarry from which the stone is taken must be stated.

Although called granite blocks, it is not insisted that they shall be a true granite; but they must be what is commercially known as a granite, and must show a record of efficient service. The result has been, that the stone block pavements laid during the past 5 or 6 years, are very good, being nearly as smooth as brick, and of course much more durable. Examples of granite paving which the writer considers very good, may be seen on La Salle St., north of Lake St.; on North Ave., west of Clybourne Ave.; on Michigan St., east of State St.; and on many other streets. Practically no granite has been laid in "The Loop" district for many years, as it has come to be recognized that it is too noisy for an office, retail, and shopping district, and property owners are willing to pay for the renewal of a quieter pavement.

A sandstone block in the proper location has many points in its favor, but, like everything else, it shows up badly when in the wrong place. It is not suitable for streets of exceedingly heavy traffic, such as "The Loop" district. It does not get as smooth or rounded as the harder granite, but it wears rapidly, and in holes. If it wore uniformly it would be ideal, as far as service is concerned, but under Chicago's enormous traffic it fails. An example of its use may be seen on Washington St., just west of La Salle St. Kettle River sandstone was used, and it has been in service about 4 years. It has been necessary to patch it with granite. If this had been laid on a

secondary business street, such as the principal street of an outlying district, or on a street where the business is principally wholesale, it would probably have given good satisfaction for many years. Even under its present conditions, it has not worn smooth, and the joints are still close and regular.

Considering quartzite as a sandstone, though it is generally called granite, the record is much better. It has a high crushing test, nearly equal to granite; is somewhat coarser; does not wear as smooth, and cuts very nicely. Much has been laid in Chicago, with good results. Most of it came from Sioux Falls, S. Dak. Stone from about a dozen quarries has been used in Chicago, but nearly half of them have been eliminated during the last 4 years, either because the granite was too soft, or was so hard that it could not be properly dressed.

It is generally recognized that a good granite pavement will outwear all others, but it is very noisy indeed, and is being relegated to wholesale and factory districts, where the traffic is of the heaviest truck character, and noise is a secondary consideration. It is also rather unsanitary, and is expensive to keep clean.

The filler is usually of mixed gravel and tar. Grout has been tried with success, as far as the pavement itself is concerned, but has been abandoned on account of traffic conditions, it being practically impossible to keep the roadway blocked for a week in order to let the grout dry.

Wood Block.—One of the most popular wearing surfaces of the last few years is obtained with preserved wood blocks, generally called the creosoted wood block pavement. Its advantages are that it is smooth, noiseless, sanitary, and very easily repaired. About its only disadvantage is that it is slippery. It has been used for many years abroad, and has given good satisfaction. In America it has to overcome the bad name given to wood block pavements by the old untreated cedar block pavements, generally laid on plank alone without any other foundation, with results which can easily be imagined. The modern wood pavement is entirely different, the blocks being rectangular, instead of cylindrical, and always laid on a good foundation.

The cycle of operations by which the block is prepared is as follows: The seasoned or partly seasoned timber is cut into 4 by 4-in. blocks from 5 to 10 in. long. These are placed in a vacuum chamber and the air is exhausted. The wood preserving oil is then admitted

to the chamber, and forced by pressure into the wood. It is desirable that the blocks shall be evenly impregnated with the preservative all through, especially if they are to be laid on a street of moderate traffic, as they will probably rot out before wearing out.

The Chicago specifications first required that 12 lb. of oil be forced into each cubic foot of timber; later, this was changed to 16 lb., as it was found that 12 lb. did not fill the fibers. Experience has shown, however, that southern long-leaf yellow pine requires from 18 to 21 lb. of oil to impregnate the fiber thoroughly, and the question then arises whether or not the wood is injured by the pressure necessary to inject the oil. It takes 6 hours for a charge, and a pressure of about 170 lb. is required to force the oil into the wood. About the same pressure is required for a 16 or 20-lb. treatment, the latter simply taking a longer time. If a lighter pressure were used, say 100 lb., the time required for the oil to impregnate long-leaf yellow pine thoroughly would probably be from 16 to 20 hours. It might be noted here that Michigan Ave., south of Van Buren St., was laid with blocks having a 10-lb. treatment some 10 years ago, and is in perfect condition to-day. This street has a heavy boulevard traffic. Only a very few openings have been made.

It has been generally specified that southern long-leaf yellow pine alone shall be used. It is quite possible, however, that this is a mistake, and that many other varieties of pine, such as tamarack, Norway pine, etc., which are cheaper, could be used, and with excellent results. It is probably a little more difficult to season them properly, but it is claimed by the tamarack promoters that it can be just as well done, that there are fewer large knots, and that there is less likelihood of checking around the circumferences of the rings of the timber. In the new specifications, several kinds of timber, such as tamarack, Norway pine, black gum, etc., are specified.

One of the troubles with blocks having a cross-section of 4 by 4 in. is the fact that, owing to the accumulated oil on their surfaces, it is sometimes difficult to tell which is the proper way to lay them. It is obvious that they should be laid on edge, for, when laid with the grain or flat, they very soon wear. In Chicago streets, therefore, in spite of rigid inspection, some of the blocks have been placed on their sides. The way to remedy this is to use a block having a different width and depth, such as $3\frac{3}{4}$ by 4 in.

It has been stated that in New York City blocks 3½ in. deep are used. In Chicago their depth is 4 in. In contrast with these figures, the practice abroad is to use blocks having a depth of from 5 to 7 in. New York City has been criticized severely by various engineers for using such shallow blocks. In a recent report* made to the City of New York, upon paving abroad, it is stated that in Paris the blocks, when badly worn, are taken up, the worn part is cut off, and the blocks are then relaid. In spite of this, their average life is only 9 years. Chicago pavements do better, for there are good creosoted pavements which are 10 years old, and, as far as can be seen, are good for 10 years more. The probable reason for the short life in Paris is stated to be the excessive amount of sprinkling.

It is also probable that another reason is the superficial treatment the wood receives. According to American ideas, the blocks are hardly treated at all, simply being given a hot bath in the preserving oil. It would seem that French engineers now recognize that this was a mistake, for, in a paper entitled "The Future of Exotic Wood as Applied to Pavements," presented to the First International Road Congress by M. Louis Mazerolle, Engineer of Bridges and Roads, of Paris, and translated for the City of New York, it is noted with approval that wood blocks, creosoted under pressure, are still in good condition after a service of 15 years.

After reading of various French methods, the American municipal engineer will certainly come to the conclusion that American construction is decidedly better, but that the French maintenance is very much superior.

Another point is the fact that the larger the blocks the more difficult it is to impregnate them completely. Therefore, when they are sawed off, a large part of the resulting surface will certainly be of nearly untreated wood. Even if the block is turned over, it will probably begin to rot in a very short time. This course of treatment, therefore, will probably prolong the life of the pavement only 2 or 3 years, and then the process will have to be repeated. Taking into account the increased cost of the blocks, the necessarily heavier treatment they must receive to impregnate them, the fact that high pressure may injure the fibers of the timber, while if a low pressure is used, the time required to impregnate the timber will be prohibitively in-

* *Engineering News*, Jan. 21st, 1907.

creased, the cost of repairing the deep blocks by sawing and the likelihood of inadequate results, it is quite probable that American practice is better. It might be mentioned that, in the same report, New York was also criticized, by implication, for the lack of depth of its granite blocks. This is not good reasoning. It is not probable that a granite pavement ever failed because of the shallowness of its blocks. Granite wears out because of the rounded edges, becoming slippery, etc., thus making a bad footing. A deep stone will certainly not make a better pavement; what is required is a better dressed block and more care in laying.

Returning to wood blocks: It has been the custom in Chicago to lay them very tight, in order that the surface may be sealed as much as possible. There has never been any trouble until the past year, when the tightness was carried to an extreme, sledges being used to make the joints as close as possible. During the winter of 1908-09 the pavement heaved. In spite of this, it is probable that no trouble would have occurred had the timber been of the proper kind. The heaving was undoubtedly caused by water absorption and then freezing and thawing. In one case tar was used as a filler, and in the other case sand.

Experiments made by the Kettle River Quarries Company* have demonstrated that, with the treatment specified (16 lb.), there is absolutely no water absorption, even when the block is left for weeks in water. There is not much expansion in wood. Trautwine states that white pine expands 1 part in 440 000 for each degree Fahrenheit. As expansion joints are always provided when such a filler as sand or grout is used, and as none is needed with a tar filler, and considering the fact that, in each case where failure occurred, the wood was supplied by a company other than that which had previously furnished it, the conclusion is inevitable that the fault was in the treatment or the timber. Though the work was not done under the writer's supervision, he is of the opinion that the fault was in the timber, in that it was of too rapid a growth, having large cells, etc., which could not be well sealed. Undoubtedly, also, in one case at least, the blocks were laid unnecessarily tight, for while they should be tightened as much as possible with bars, they should not be pounded to refusal. This point is taken up later.

* Discussion by Walter Buehler, Assoc. M. Am. Soc. C. E., at the meeting of the Illinois Society of Engineers and Surveyors, February, 1909.

In Chicago, the Kreodone-Creosote oil, produced by the Republic Creosoting Company, has always been the standard, and the results have been satisfactory. In a paper presented before the Illinois Society of Engineers in 1908, John B. Hittell, M. Am. Soc. C. E., Chief Engineer of Streets, Chicago, shows that a large percentage (about 9%) of the oil of a half block volatilized in the first three days it was exposed in the even temperature of a closed room. That more than this is lost on the street is almost certain, and the action is very evident from the appearance of the blocks piled on the street and waiting to be used. In the strong sunlight they very soon begin to check badly in all directions, and the action continues until they are laid in the pavement, filled, and covered with sand. After they are in service, there is little apparent loss.

It was early recognized that to use as a standard the treatment advocated by any one company was anything but scientific, but at that time the city had no adequate laboratories in which to try out and analyze the different processes. Since the installation of an asphalt laboratory in 1907, however, it is in better shape, and the specifications have now a more scientific basis. The principal points are as follows:

The quantity of oil used is 20 lb. per cu. ft., but it is the writer's belief that, for the reasons previously stated, this is unnecessarily large. The oil must be a pure, coal-tar product, free from any adulteration; it must not contain any petroleum, nor more than 5% of matter insoluble in benzine. Oil obtained or partly obtained from water-gas tar, or oil tar, will not be accepted.

The specific gravity of the oil shall be at least 1.1 at 25° cent.

In the matter of volatilization of the oil, which, as has been stated before, was found to be a very serious one, the following provisions have been made:

Up to 150° cent.	Nothing must come off.
" " 210° "	Must not volatilize more than 5 per cent.
" " 235° "	" " " " " 15 " "
" " 315° "	" " " " " 40 " "

It is the writer's opinion that it will be exceedingly difficult, if not impossible, to enforce rigidly these percentages, but, undoubtedly, an attempt should be made to approach this standard.

Another very important point, which has been mentioned before, is the large size of the cells and hence of the annual rings of the timber; this is also noted in the specifications, and the provision is inserted that, in long-leaf yellow pine, the number of annual rings shall be not less than seven per inch, measured radially from the center. This provision will bar out first-growth timber which has had too rapid a growth, and in which the cells are so large that it is difficult to seal them. As previously mentioned, one of the apparent failures was probably due to the freezing and thawing in the cells of the timber.

One of the troubles with creosoted block pavements has been caused by the brow on streets having car tracks. Such streets, ordinarily, have more than the average amount of traffic, as they are generally the business streets. The Chicago specifications provide that the blocks shall be placed on a sand cushion of 1 in., but even if the greatest care is taken with the concrete, it is not possible to get an absolutely uniform surface, and the sand bed will vary from ½ to 1½ in. As the blocks must be rolled after they are laid, the sand bed must be graded so that when first laid the blocks will project somewhat above the car tracks. When the street is rolled the blocks are forced down, and afterward the traffic pounds them down still farther. In the course of time, if the blocks have not been set high enough, or the sand cushion is too deep, they are pounded down until they are considerably below the edge of the rail, making an ugly bump in the street, and a depression in which water will stand along the edge of the rail. Fig. 2 shows this condition; it should be noted that the shoulder, *A*, is too high, and *B* is too low. Water collects at *C* and disintegrates the pavement.

On the other hand, if the blocks are set so high that they never pound down to the top of the rail, the pavement along the brow will fail in a few years from brooming along the edge, thus tending to tip at the rear end of the block. Fig. 3 shows this condition. Both *A* and *B* are too high, the block is ground and distorted at *C*, and the accumulation of water at *C* tends to destroy the pavement.

In Figs. 2 and 3, *C* is the controlling point, and must be higher than any point beyond it.

Either of these cases can be avoided only by the greatest care in shaping the concrete foundation at the brow and shoulder. If the blocks are 4 in. deep, and 1 in. of sand bed is used, the concrete should be 4¾ in. below the top of the rail. Sand 1¼ in. deep will compact to 1 in.

The block will then be ½ in. above the rail when first laid. Rolling and tamping will reduce this to ¼ in., and the traffic, after a shorter or longer time, will pound the brow down to the rail level.

It seems to be a custom in Chicago and elsewhere to require that the courses be laid at an angle with the line of the roadway, generally, 45 degrees. The result is an enormous amount of waste, because every course must be ended and finished by a skew block, and the regular blocks must be cut at an angle to make this skew. These skew blocks, on account of their sharp, triangular ends, are also particularly susceptible to wear, and must be and are a point of weakness. It seems to the writer to be the height of absurdity to continue this 45° or similar angle. It is simply following the custom of brick paving, without any of the same reasons. In brick paving the joints are comparatively wide, therefore it probably saves a good deal of wear on the edges to lay the bricks at such an angle, but a preserved wood pavement is as smooth as asphalt, has practically no open joints, and will certainly be cheaper and better if the courses are laid at right angles to the line of the street.

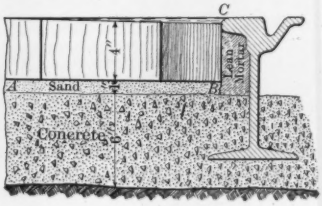


FIG. 2.

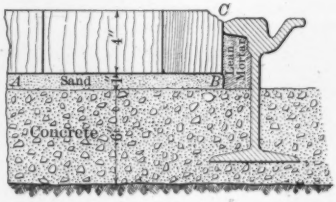


FIG. 3.

Asphalt.—Asphalt is undoubtedly the most popular pavement in the larger cities of the country to-day. Approximately 50% of all the pavements now being laid in Chicago are asphalt. The reasons, of course, are its noiselessness, cleanliness, ease of repair, good looks, and comparative cheapness. It costs less than brick, only 60% as much as creosoted block, and about 50% more than standard macadam. It is, however, probably the most technically difficult pavement to lay, and should never be laid without expert advice.

As a material, it has been abused in past years more than any other. It has been laid by irresponsible firms, without adequate advice, has been laid on streets for which it was not adapted, and the result

has been many failures. The authorities, however, have come to the understanding that it needs the best of supervision and inspection, and, under such conditions it has become nearly the ideal paving material. Formerly it was the custom to accept the guaranty as being all that was necessary to get good results. The city engineer had very little knowledge of the material, beyond that imparted by the companies doing the work. It is different now; Chicago, like other cities, has established an asphalt laboratory, and samples from each job are examined daily by a competent chemist.

The modern asphalt pavement is composed of the foundation, the binder course, consisting of small crushed stone, of 1 in. in diameter and less, mixed with asphaltic cement, intended to give body to the top; and the top or wearing surface, consisting of asphaltic cement and fine sand and a filler of Portland cement.

The specifications were formerly very general indeed, no percentage of bitumen was specified, nor was any attempt made to regulate the manner of preparing the refined asphalt, the flux (or oil to be mixed with the refined asphalt, to render it a workable cement), or the sand grading. Now all this is looked into closely. It is specified that crude natural asphalt shall not be heated to a temperature of more than 400° Fahr. in order to drive off the water. It is also specified that at least 98½% of the bitumen which is soluble in cold carbon bisulphide shall be soluble in cold carbon tetrachloride.

Mr. L. Kirschbraun, Asphalt Chemist for Chicago, shows by detailed experiments* just why this provision was inserted, as it was objected to by some of the native asphalt companies. Suffice it to say that he demonstrates that the carbenes (which is a term applied to those asphalt bitumens which are more soluble in carbon bisulphide than in carbon tetrachloride) are an evidence of lack of care in the preparation of asphalt from the native asphaltic oils, and for that reason the clause is justified.

Formerly the use of Texas or California asphalts for the wearing surface was not permitted, but of late years it has been demonstrated that if these heavy oils, having an asphaltic base, are prepared properly, they are the equal of any of the Trinidad or Bermudez asphalts. Therefore the Chicago specifications now provide for the use of Cali-

* In an article in *The Municipal Engineer*, December, 1908.

fornia asphaltic petroleum, distilled at a temperature of not more than 600° Fahr., giving a residue not harder than 40° penetration at 77° Fahr., by the Dow machine. The requirement as to carbon tetrachloride, of course, applies to this asphalt in common with the others, together with a test to show that most of the light oils have been driven off in distillation. The requirements are that the sample tested shall not lose more than 5% by weight, nor shall the penetration be reduced more than 50% when heated for 7 hours at a temperature of 325° Fahr.

The fluxing material is also carefully specified, and as carefully examined for penetration, and for water, coke, and other impurities. The whole idea is to prepare an asphaltic cement which will not become too soft in summer or too brittle in winter. Equal care is taken with the sand grading; the sizes of the grains must be such that practically none shall pass a 200-mesh sieve, all shall pass a 10-mesh sieve, the bulk of it shall pass a 40-mesh sieve, while a smaller part shall pass an 80-mesh sieve.

The binder, also, is now what is called a tight material, as opposed to the old open binder. In the tight mixtures the voids are nearly filled with small stone or torpedo sand, in the open binder this stone was of nearly uniform size, about ½ in. The result of this was many so-called "fat" spots, or a concentration of the asphaltic cement, which had a tendency to rise through the wearing surface and make a weak place in the pavement, and this invariably wore into a hole. The tight binder mixture prevents this, as the asphaltic cement is distributed more uniformly through it. It also has an advantage which is immediately apparent. It may be teamed over, without breaking up under the traffic. This is necessary because the material for the top or wearing surface must be delivered in wagons over it. The old open binder very often broke up under such traffic, and then became no better than the old cushion coat used 20 years ago. It is too soon as yet to see the improvement in the wear of the pavement, as thus inspected and prepared, but the writer thinks he is safe in saying that not a single pavement laid during the past 2 years, when this care has been taken, has cracked or shown signs of failure.

Asphalt, according to experience in Chicago, is not suitable for streets having the heaviest commercial traffic, such as those in "The Loop" district, as the best pavements laid thus far have shown a

tendency to wave and wear rapidly. As an example of this, six blocks on State St., from Randolph to Jackson Blvd., may be cited. This stretch was laid by one of the most responsible companies in the business. It employs a chemist constantly to look after the mixtures. Part of the pavement was under a 10-year guaranty, and part under a 5-year guaranty. On both stretches the guaranty will expire within a year. This street has an enormously heavy traffic, being in the heart of the city's retail shopping district. A repair gang is kept busy patching almost constantly, and the surface is quite wavy. On account of the character of the traffic, every effort was made to get a perfect construction, but, while the pavement is kept in fairly good condition with constant repair, the surface cannot be called a success on such a street.

Again, property owners, in their enthusiasm for a nice, smooth pavement, have called for asphalt pavements on streets where the traffic is so light as to be almost nothing. Here, again, the result has been bad. Lack of traffic, apparently, is nearly as fatal as too much. The surface cracks, shrinks, and dries up, and in three or four years is a mass of unsightly cracks. Between these extremes the results have been best. Many examples of this may be shown: La Salle Ave., from Chicago Ave. to Clark St., which is 15 years old, has a fairly heavy traffic, and, although it is rather badly worn now, 80% of its surface is in good condition. Dearborn Ave., between the same limits, is 13 years old, and is in excellent condition. Indeed, one block of this is more than 25 years old, and can hardly be distinguished from the remainder. Both these streets have been repaired within the last year.

As a general rule, however, the impossibility of making repairs, except with a special plant, makes asphalt a not altogether desirable pavement for a small town, unless that town is very near a large one. The amount of work to be done in such a town is not enough to pay the contractor to keep a permanent plant there, and therefore he will make repairs but once a year, if possible, and bad places in the pavement may go several months before being repaired.

In Chicago nearly all the various brands of asphalt, both native and imported, such as Trinidad, Bermudez, California, Texas, etc., have been used, and under proper supervision they are all apparently good. For some of them too short a time has expired to be sure of results, but none has shown signs of failure as yet.

While streets may be seen in Chicago where asphalt has been in service for 20 years or more, it is probable that, after about 10 years, there is practically no "life" left in the wearing surface. Practically all the oils, both light and heavy, have been volatilized by that time, and a sample of the top may be disintegrated between the fingers, producing nothing but clean sand, and leaving absolutely no stain on the fingers. The blackness may be seen in small grains, and is probably the "coke" of the old asphaltic cement. The writer has made this experiment many times with the same result. On streets with light traffic the pavement will probably be good for some years after that, but on streets with heavy traffic the surface goes to pieces rapidly when this stage is reached, and even constant patching will not keep it in good condition. An example of this may be seen in North Clark St., between Kinzie and Division Sts. This was paved in 1897, has a very heavy traffic, and has been repaired every year, for the past few years. In the fall of 1908, it was thoroughly repaired, but at present (April, 1909) it is again in bad shape, new holes constantly appearing.

Bitulithic pavement, nearly allied to asphalt, has become popular of late years, and has much to recommend it. It is patented, and for that reason cannot be laid in Illinois by special assessment. In Chicago, however, it can be and is laid by the park boards, which do not work under the same law. It has been very successful. It consists of asphaltic bitumen mixed with a carefully-graded aggregate of crushed stone. The mixing and grading of the stone are done by special patented machinery, and, when the mixture is delivered on the street, it is spread and rolled like ordinary asphalt. On the surface is spread a flush coat of bitumen, and chips are then spread over this flush coat and rolled in. The result is a fine, even surface, not as smooth as asphalt, but in other respects very similar. On account of the fact that the city cannot legally lay such a patented pavement, none has been laid under the writer's supervision, but from observations of pavements put down by the park boards, his opinion of it is very high. One stretch of ¾ mile, on Sheridan Road from Irving Park Blvd. to Byron St., which he has had under observation, has been down 4 years, and has been very satisfactory. The street has an exceeedingly heavy boulevard traffic, but there have been practically no repairs, the surface is in excellent condition, and has not cracked, or become slippery. The cost of the bitulithic pavement is slightly greater than that of

asphalt, and, if equal precautions are taken in the selection of the bitumen, there is no reason why its life should not be as long, if not longer.

Another adaptation of this same principle—which may or may not be an infringement on the bitulithic patent—is the so-called "Mineral Rubber" pavement, now being laid by the park boards in Chicago. It consists practically of run-of-crusher stone, 1½ in. and less, mixed with asphalt. No particular attempt is made to grade the stone, and the ordinary brands of asphalt may be used. A flush coat of asphalt is spread, and the top is sprinkled with torpedo sand. In Chicago it has been mixed with portable mixing machines and laid on the street in sections across the roadway. These sections are about 8 ft. wide, and are separated by wooden strips. The surface has been very good, smooth and not slippery, but is somewhat uneven, due probably to lack of rolling. The writer is opposed to the strip method, and more in favor of continuous construction. Joints are always points of weakness, and he sees no good reason for making an exception in this case.

Miscellaneous Pavements.—Various kinds of concrete pavements have been laid with some success in Chicago. These consist essentially of a bottom course of about 5 in. of ordinary 1:3:6 concrete, and a second course of richer concrete, laid before the first has set. The whole is then finished with a flush coat of rich mortar, which is smoothed off and grooved into rectangles of about 5 by 10 in. The greatest care must be taken to get proper expansion. A system of streets in the vicinity of the Sears, Roebuck Company's plant on the West Side of Chicago, only 3 years old, is very badly cracked, both longitudinally and transversely, on account of expansion. In places this action has nearly overturned the curb. Wide cracks extend down the center of every street, and the pavement is in bad shape. Plate I is a photograph of one of these streets. No comment is necessary. A similar pavement, laid on Ohio St., near the Lake Shore, which is only 1 year old, but in which adequate expansion joints were provided, shows no signs of failure.

These pavements are very desirable in many ways, being easily cleaned and easily repaired. They have been used in several alleys. The surface has not worn too well, in some places, under heavy traffic. One alley in the rear of the Lexington Hotel and another at 31st St.

and Wabash Ave., have shown decided signs of failure, the joints being battered down and the surface disintegrating, although they have been finished for less than 1 year. They were laid in two courses, a bottom course, probably of 1:3:6 concrete, and a top course of rich mortar, probably 1:2. These are extreme examples, and the pavement should not be condemned on these alone. They were not laid under strict supervision, being done by what is called private contract.

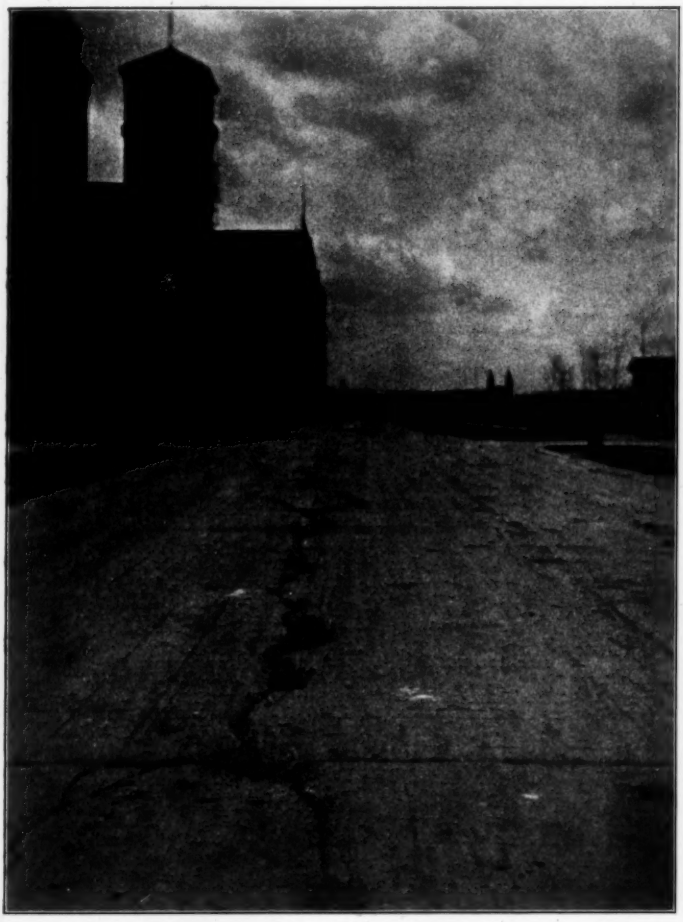
Various types of patented pavements, of wood, concrete, etc., have been tried from time to time, but the trial pavements are too limited in area to enable one to comment intelligently on them.

TABLE 1.—Miles of Pavement of Various Kinds Laid in Chicago by Special Assessment, from 1903 to 1908, and Average Prices per Square Yard.

Year.	Asphalt.		Brick.		Creosoted Block.		Granite.		Macadam.	
	Miles.	Average price per square yard.	Miles.	Average price per square yard.	Miles.	Average price per square yard.	Miles.	Average price per square yard.	Miles.	Average price per square yard.
1903..	49	$2.46	7	$2.65	0.03	$3.25	3.5	$4.33	20	$1.38
1904..	43	2.50	5.5	2.91	0.57	3.52	4.6	4.06	31	1.38
1905..	48.7	1.985	6.6	2.26*	0.20	2.65†	4.0	3.60	38.9	1.22
				2.62‡						
1906..	52.4	1.88	5.1	2.38*	0.75	3.23	3.9	3.59	43.5	1.15
				2.64‡						
1907..	57	2.17	3.25	2.26*	0.90	3.78	5.5	3.93	18.9	1.21
				2.62‡						
1908..	38	2.09	10.6	2.35*	4.7	3.93	6.5	4.06	16.7	1.25
				2.51‡						

* All streets.
† One job, blocks laid on old macadam.
‡ All alleys.

Table 1 shows the various classes of pavements laid each year by special assessment, and the average prices per square yard. Asphalt is by far the most popular in the city districts. In the outlying districts, macadam is naturally the most popular, on account of its low first cost. The prices include such grading as is necessary to prepare the subgrade, where a street is being repaved, but do not include any large amount of grading in any case. The price per square yard includes the pavement complete, that is, the foundation and the wearing surface, but not the curbs, sewer adjustments, etc. All pavements are under a 5-year guaranty except macadam, which is under a 2-year



CRACKS IN CONCRETE PAVEMENT IN A CHICAGO STREET.

guaranty. It will be noticed that the price of brick is very high. This is because brick has been used almost exclusively as an alley pavement in recent years. All the foundations (except macadam) are of Portland cement concrete, 6 in. in depth. The macadam consists of an average of 12½ in. of material, 9 in. being limestone in two courses and 3½ in. being granite in one course.

TABLE 2.—QUANTITIES OF PAVEMENT IN CHICAGO AT THE PRESENT TIME, 1908.

	Miles.	Approximate Yardage.
Asphalt	416	6 955 000
Asphalt blocks	2	30 000
Rock asphalt	0.6	10 000
Brick	107	1 260 000
Cedar block	452.50	8 100 000*
Concrete	3	30 000
Creosoted block	7.25	130 000
Granite	63	1 260 000
Medina stone	1.15	20 000
Macadam	552	9 000 000
Novaculite macadam	2	32 000
Slag macadam	6	96 000
Total	1612.5	26 915 000

* Practically all the cedar, some 8 000 000 sq. yd., is now worn out and useless; probably 90% of the remainder is in fairly good condition, and 10% is in bad condition.

DISCUSSION

Mr. Crosby.

W. W. CROSBY, M. AM. SOC. C. E. (by letter).—The writer has found this paper extremely interesting, and desires to call attention to the following points:

Under the heading, "Wearing Surface, Brick Paving," the author, in expressing his opinion regarding the allowable limits of loss in the rattler, states that "bricks which lose more than 20% of their weight are rejected." Will he kindly state whether or not, in his opinion, it is sufficient to limit the average loss to 20%, or whether an attempt should be made to prevent too great a variation between the minimum and maximum losses by specifying the maximum variation to be allowed between the extremes? The writer believes that such a limit in variation should be specified, in order to avoid lack of uniformity in the surface.

Under the heading, "Stone Block," the author states that preference is given to certain quarries which, as shown by experience, have furnished satisfactory blocks, and apparently no tests of the blocks themselves, other than that of actual use on the streets, have been, or are, required. Does this practice result in limiting competition unnecessarily, or in excessive high cost, through a combination of the operators of the preferred quarries? Possibly a test similar to the customary rattler test of macadam materials could be made on material offered for stone-block work; and, by specifying the maximum loss necessary to reject a block which would wear too rapidly to be economical, and a minimum loss lower than which a block might be expected to wear rounded and slippery too easily, one might avoid conditions, which, in some localities, might be objectionable.

Under the heading, "Wood Block," the author suggests certain limits for the allowable volatilization of creosote oil. In the writer's opinion, the 5% and 15% limits at 210° and 235°, respectively, will be raised, for the present at least, on account of existing conditions, to 10% and 20%, respectively.

Under the heading, "Asphalt," the author states that:

> "Asphalt, according to experience in Chicago, is not suitable for streets having the heaviest commercial traffic, such as those in 'The Loop' district, as the best pavements laid thus far have shown a tendency to wave and wear rapidly."

It would be enlightening and interesting if, where such apparent failures have occurred, a detailed statement could be made of the composition of the pavement, both as regards the properties of the bituminous material and the character and proportions of sizes of the mineral aggregate. It will be generally agreed that many failures,

such as the author refers to, are due to defects in one or the other of these factors. The author seems to be of the opinion that about ten years is practically the limit of the life of asphalt pavements. The writer admits that many have failed in even much less time, but believes that, with proper materials, asphalt pavements have been and can be constructed which will have a life of several times the period mentioned. Mr. Crosby.

In reference to concrete pavements, it would be interesting if the author could state any facts concerning the cost of repairs to such pavements. He mentions that the surface has not worn well in some places, but it is not clear whether the defects have been caused by the use of improper materials or methods, or by inequalities in either. The writer's experience is that it is very difficult, if not impossible, to secure a concrete surface which will wear evenly under traffic, frequently because of the unavoidable variation in the broken stone used and in the proportions of the mixture itself.

The work of keeping such pavements in satisfactory repair is extremely difficult, as well as high in cost.

E. H. THOMES, ASSOC. M. AM. SOC. C. E.—Highway construction and maintenance, while one of the oldest branches of civil engineering, has only recently begun to receive the serious consideration and thorough investigation by engineers which it deserves. There is still a wide field for new and advanced methods which would effect a large ultimate economy in the millions of dollars being expended for highway improvement. Mr. Thomes.

During the summer of 1909 the speaker made a paving inspection trip through twenty of the principal cities between New York and Chicago, and a few of the impressions gained may be of some interest.

In the past few years Chicago has made notable progress in street improvements, as has been so ably presented in Mr. Green's paper.

A matter which should receive the early and careful consideration of the municipal authorities is the complete establishment of street lines and grades for the entire area of the city, so that, as various improvements are made, they may conform to a comprehensive and well-planned street system. This should include roadway and sidewalk widths, together with explicit rules for determining the auxiliary grades of the roadway crown, the gutter line, the front and back edge of the curb, the location and slope of the paved footwalk, whether of cement or stone, the slope of grass plots, the determination of grades at all these intersecting lines, the radius of the curb at intersections, the drainage, the location of trees, areaways, stoop and house lines, etc.

The following is the ordinance in New York City for street widths in future improvements:

Mr. Thomes.

NEW YORK CITY ORDINANCE.

The roadway widths of streets shall be such as to give a clear space between curb lines as follows:

(*a*) For streets less than 20 ft. wide and used for vehicular traffic, the width of the roadway shall correspond with the street width, less the space occupied by the curb.

(*b*) For streets having a width ranging from 20 to 50 ft. and not occupied by a railroad, the width of the roadway shall be 60% of the total width of the street.

(*c*) For streets having a width ranging from 50 to 60 ft., and not occupied by a double-track railroad, the roadway shall have a width of 30 ft.

(*d*) For streets having a width ranging from 60 to 66 ft. 8 in. and not occupied by a double-track railroad, the width of the roadway shall be one-half of the total width of the street.

(*e*) For all streets having a width of over 66 ft. 8 in., except those portions of Fifth Avenue and of Forty-second Street, Borough of Manhattan, concerning the treatment of which a resolution was adopted by this Board on December 18th, 1908, the roadway width shall be 80% of the street width less 20 ft.; provided, however, that if the street is occupied by a double-track railroad the minimum roadway width herein prescribed for such railroad shall be required.

(*f*) For streets in which there is a single-track railroad, the minimum roadway width is to be 30 ft.

(*g*) For streets in which there is a double-track railroad, the minimum roadway width is to be 40 ft.

The curb corners at street intersections where the interior angle is 30° or more, shall be turned with a curve having a radius of 5, 6, 8, 10 or 12 ft., this being determined for each case as the nearest of these dimensions which would represent 10% of the width of the wider street, provided, however, that in case the interior angle is less than 80° the radius shall not be less than 20% of the distance between the building line corner and the point of intersection of the curb tangents.

For intersections where the interior angle is less than 30° a tangent shall be inserted in the curb line at the corner at right angles to the line bisecting the said interior angle, and at a distance from the building line corner equivalent to the width of the wider sidewalk of the intersecting streets, the said distance being measured along the bisecting line, the curves to connect this tangent with the curb lines otherwise provided for shall each have a radius of 6 ft.

The roadway shall be centrally located between the street lines, and for streets having a width of 20 ft. or more, the remaining space on each side of the roadway shall be designated as the sidewalk.

No encroachment shall be permitted upon any roadway unless authorized by a franchise.

No encroachment of a permanent nature shall be permitted upon the sidewalk space of streets owned in fee, or shall be hereafter permitted upon an easement street, between the elevation of the curb and a horizontal plane 10 ft. above the elevation of the curb.

Mr. Thomes.

If Chicago's 2-ft. and 4-ft. street corner curb radii were made larger, it would improve the general appearance and facilitate vehicular traffic. On the other hand, the radius of 15 ft. and greater on the average street is likely to encourage too high a speed in vehicles turning the corner, and to endanger pedestrians crossing, also to curtail the sidewalk space.

There are many advantages in a street system having the four intersecting street curb corners all at the level of the established grade, with a straight curb grade between adjoining streets or only one intermediate change of grade, eased off by a vertical curve and with the proper grade to drain well. The depth of gutter may then be uniform for all pavements, the crown or elevation of the center of the street being varied to meet the various conditions. In the case of steep or very flat grades, skew intersections, car tracks, permanent improvements, etc., it becomes a difficult problem to solve satisfactorily.

The most common form of street surface is the parabolic curve, with a crown of $\frac{1}{60}$ of the width of the roadway, and a 5-in. or 6-in. depth of gutter. In recent years the tendency has been to increase the crown of sheet asphalt and other pavements from $\frac{1}{100}$ and $\frac{1}{80}$ to $\frac{1}{72}$ or even $\frac{1}{50}$ of the width of the roadway. Smaller and more frequent sewer inlets are constructed, and the depth of the gutter is reduced.

Stone curbing should be set in concrete made with small stone, but it appears that an unnecessary depth of curb and an undue quantity of concrete is used in some places, the curb being 24 in. deep, with 6 in. of concrete all around. It would seem that a depth of 16 or 18 in. and 4 or 5 in. of concrete below and around the curb would be ample in most cases. In wet subsoil, a drain of cinders, gravel, broken stone, or tile should be placed below the curb and under the pavement where necessary. These drains should be connected with the sewer inlets at every low point. In sandy or other well-drained subsoils, and in closely built-up sections with deep cellars and sewers, sub-drainage is not needed.

The character of the curb will depend on local conditions. Concrete curb is favored in many places on account of its cheapness and pleasing uniform appearance. A metal-edge reinforcement is an improvement, but as most of those on the market are patented and are expensive, its use is not justified in some cases. A metal-edge reinforcement on both concrete and stone curbs is advisable on street corners where the traffic is heavy. Care should be taken not to bend the metal nosing, as it is difficult to keep it to the true curb line.

The combined concrete curb and gutter is satisfactory in some suburban districts, or with a macadam or a bituminous pavement, but it does not meet with general favor in large cities where the traffic is heavy. It should be constructed in sections of about 6 ft., of well-rammed fine concrete on a firm cinder base, and should have a wearing

Mr. Thomes.

surface finish of mortar composed of fine granite screenings or other hard grit. Considerable combined concrete curb and gutter has been laid in Chicago, and has been fairly satisfactory. It is cheaper than stone curb; it reduces the area of pavement to be paid for; it is easier to construct a smooth and varying gutter line; and it avoids the disintegrating effect of water in an asphalt gutter; but after a few years under heavy traffic it wears out at the junction with the pavement, and it is very costly to repair the pavement or the gutter.

If more attention is paid to draining, rolling, compacting, and surfacing the subgrade, a considerable saving may be effected in the quantity or quality of the concrete or stone foundation and in the maintenance of the wearing surface. In Cleveland, which probably has as good pavements as any city in the United States, special care is taken in puddling, rolling, and preparing the subgrade. The subgrade and the sand bed for block pavements are brought to the proper surface by the use of a right and a left iron-faced wooden templet moved by iron handles. The templets are about half the width of the roadway. They have a 4 by 4-in. notch in each end, which fits over a 4 by 4-in. timber placed to grade along the center of the street and along each curb. In Rochester a trussed templet, the full width of the roadway, is drawn along on the curbs, with the bottom of the templet at the proper elevation to give the true contour. In other cities the proper depth of concrete, sand bed, etc., is obtained from ordinates below a string stretched across the curb, or from the engineer's stakes, or by tees, or by stakes set at close intervals by these methods. Instead of using wooden stakes, it would seem advisable to use flat-headed iron spikes which can be easily driven, pulled up, and used over and over again. Too much care cannot be taken to obtain uniform depth and a true curvature of the different courses.

While a rolled-stone macadam or gravel base foundation is satisfactory in many places, the general practice in most of the larger cities is to use a concrete foundation. The good results from a stone foundation are probably due to the increased rolling. If the subgrade is properly rolled and drained, and the concrete is well compacted, better results will probably be obtained from concrete. A cement-grouted rolled-stone foundation similar to the "Hassam" pavement, would seem to offer some advantages. A roller-tamped stone foundation, similar to the improved "Petrolithic" pavement, may be suitable in many places.

Brick pavements, while quite popular in the Central States, have not been used very extensively in New York City. They have not proved satisfactory under the heavy traffic, and they cost more in this locality than asphalt pavements which give better satisfaction. Where traffic can be kept off, a good cement grout filler is preferred. Tar filler is used more than asphalt filler in most places, because it is cheaper. A mixture of tar and asphalt might prove more suitable

in many cases, all conditions considered. With grouted joints, an expansion joint along the curb is advisable in practically all cases; but, in the case of frequent transverse tar or sand expansion joints, the remedy is worse than the disease, as the pavement begins to disintegrate at all these joints in a short time, and the results appear to be more objectionable than the fine cracks or occasional wide cracks which may develop where no transverse expansion joints are used. In most cases the cross expansion joints are not advisable, especially for stretches of less than 1 000 ft., or where the bricks are heated and expanded, as they usually are, when laid in the summer. Where provision for expansion is deemed necessary, the wood block expansion joint, as developed in Detroit in 1908, gives promise of being more satisfactory than the usual bituminous transverse joint. Two courses of creosoted, 4-in., long-leaf pine, paving blocks are laid across the street about every 100 ft. They are laid, rolled, grouted, and treated in exactly the same manner as bricks. They have been in use for more than a year, and seem to take care of the expansion without developing any noticeable cracks, either at or between the wood blocks. The wood blocks will probably wear about as well, if not better, than the adjacent brick, as they are likely to wear more uniformly than the brick. Mr. Thomes.

While the speaker was in Chicago, the wood block pavement in front of the Auditorium on Michigan Avenue, south of Van Buren Street, referred to by Mr. Green, was being removed to permit the widening and change of grade of the new Michigan Avenue (Park) Boulevard. Only a small strip of the wood block pavement remained, but it was in excellent condition, and the blocks showed a wear of only about ¼ in. in the ten years it had been laid. Further authoritative detailed information about this piece of pavement would be very desirable. It was laid on a mortar bed, which is the New York City practice. Most of the wood block pavements in the West are laid on a sand bed and rolled like brick. In the latter case the general appearance of the surface is usually better, due to the rolling, but a mortar bed does not shift, and it holds the blocks better, as it is not so readily affected by water getting under the blocks. The author's statement that with a 16-lb. treatment there was absolutely no water absorption, requires further explanation. Under ordinary conditions there is usually some drying out and some water absorption, with contraction, swelling, and expansion of the wood block. This depends on the kind, quality, size, closeness of grain, extent and method of seasoning, etc., of the timber, on the temperature, pressure, duration and method of treatment, and the character of the preservative used; also on the method of pavement construction, the joint filler, the amount of traffic, the grade, the exposure to the sun or to moisture, and to various other local conditions. Wood block is like sheet asphalt in that it is better under sufficient traffic to iron out and seal the wearing surface.

Mr. Thomes.

The specifications prescribing the number of annual growth rings per inch for long-leaf yellow pine is a step in the right direction. Only those who have had actual experience can realize the difficulties of grading Southern yellow pine satisfactorily to all parties. When sawed into lumber it seems to be practically impossible to distinguish some slow-growth short-leaf and other pines from some quick-growth long-leaf pine, or to distinguish second-growth timber positively, and the most feasible method of determining their classification appears to be by the average number of annual growth rings per inch. This matter is not yet sufficiently developed in the present commercial practice, but it will probably be adopted in due time. There are a number of details to be taken into consideration, and it will require time to work out the necessary practicable rules. The manufacture and treatment of creosoted wood blocks should be carefully inspected at the plant. It is desirable that the timber be inspected in the stick before it is sawed into blocks.

There is considerable difference of opinion as to the timber, the proper specific gravity, and the character and amount of wood preserving oil, the methods of seasoning and treating, and also the tests of the treated wood block. These matters require further investigation.

There seems to be no advantage in the common Western practice of laying the wood blocks at an angle of from 20° to 45° with the curb, and with several longitudinal courses of blocks along the curb. It would seem that equal if not better satisfaction is obtained with all the blocks at right angles to the curb, and with an expansion joint along the curb. Ordinarily, no transverse expansion joints are advisable. With all the block pavements, better results seem to be obtained with all the courses at right angles to the curb, except at intersections or along car tracks. With sheet asphalt two or more courses of brick, wood, asphalt block, sandstone or granite blocks are usually laid parallel to the rail. The blocks should be laid in a mortar bed. The joint filler will be governed by local conditions.

A matter which deserves the concerted action of city engineers and paving contractors is the standardization and adoption of uniform specifications and tests for stone block pavements, similar to those adopted for brick. If two or three uniform standard specifications for stone block pavement could be adopted, they would meet the requirements for practically all conditions. The stone quarries could work their plants to better advantage, and the cost of the blocks would be reduced. The stock blocks could be kept on hand, and their delivery facilitated. At present there is such a diversity in the various stone block specifications that it is impracticable for a quarry to keep sufficient stock on hand, and the construction work is often delayed in obtaining the proper blocks. The different specifications have limits for the depth of block varying between 4 and 8 in. As a depth of from

Mr. Thomes.

3 to 4 in. has been found to be sufficient for brick, wood, and asphalt block, a depth of 5 in. should be sufficient for stone block, even under the heaviest traffic, because the pavement will probably fail on account of the unevenness of the surface, due to the non-uniformity of the stone and the concentration of traffic, or because of poor foundation, before it will fail on account of lack of depth of the stone blocks. The character of the stone block will depend on traffic and other conditions. If more attention is paid to securing a good quality of stone block pavement with a smooth surface and close joints, it will wear much longer and more uniformly. Although, with this kind of pavement, there will be an increased cost, due to the better quality, dressing, and laying of the stone block, there will be a decreased cost in transportation and excavation, and closer joints can be secured more easily with a shallower block. Considering the decreased cost of maintenance, ease of traction, saving in wear and tear of vehicles, diminished noise, improved general appearance, and other advantages, a good stone block pavement will be cheapest in the long run.

By co-operation among the various engineers and contractors to secure the essentials without unnecessary refinements, pavements can be improved without much increase in cost. While it is advisable to improve the general condition of stone block pavements, care should be taken not to go to extremes. The new granite block specifications of Newark, N. J., contain a number of improvements, but go to uneconomical extremes, for instance:

"The blocks must have nearly plane surfaces for all of the faces of each block so that when a straight-edge of the full length of the block is applied in any direction upon said faces, there will be no depressions or bunches which shall deviate more than $\frac{3}{16}$ of an inch from said straight-edge."

Much of the new stone block pavement in Chicago consists of a reddish Wisconsin granite, which has a smoother head and seems to be of a better quality than most of the Eastern granites.

While actual long-time service is the best test of any stone block, physical and chemical tests would be of great advantage, if proper tests can be evolved, which will give fairly reliable indications of the actual wearing qualities on the street. Considerable work along these lines has been done by the United States Office of Public Roads. Further information relative to actual wear and to tests on stone paving blocks would be of interest, also any experience with rolling stone block pavements. It was stated in Cleveland that the Medina sandstone block pavements were rolled in the same manner as the ordinary brick pavements. This appears to have some advantages.

A grout joint filler for stone blocks is preferred where the traffic can be diverted. A bituminous filler is used where the street cannot be kept closed or where the pavement is likely to be opened up fre-

Mr. Thomes.

quently. A sand joint filler is also used. The grout varies from 1 part of cement to from 1 to 4 parts of sand.

The asphalt block pavement is worthy of further investigation by municipalities. Some of the early asphalt block pavements proved defective; but the present improved asphalt block, which is carefully manufactured under a pressure of 6 000 lb. per sq. in., is extensively used in New York City, and, on the whole, it is giving good satisfaction. It is somewhat more expensive than sheet asphalt, but is less slippery and is especially suitable for steeper grades and for places at a distance from an asphalt plant, as it can be laid and repaired easily. The asphalt blocks are 12 in. long, 5 in. wide, and 3 in. deep. They are laid upon a concrete foundation in a $\frac{1}{2}$-in. mortar bed, with close joints filled with sand. The asphalt blocks are composed of asphaltic cement, crushed trap-rock passing a $\frac{1}{4}$-in. screen, and limestone dust. The block has a specific gravity greater than 2.5.

A cement concrete wearing surface does not appear to meet with much favor in the larger cities. It is suitable for alleys and streets with light traffic, but it is questionable whether it will be satisfactory under heavy traffic in the large Northern cities. It has some advantages, and, with further improvements, it may prove satisfactory in many cases.

Kentucky rock asphalt pavement has met with success in some places and with failure in others; it is difficult to obtain uniform material, or bitumen of the proper amount and character, and it wears unevenly in many cases. Other bituminous pavements have given better satisfaction.

The present sheet asphalt specifications and methods of supervision in Chicago rank with those of any city in the United States. The method of control of asphalt construction is similar to that of Washington, with some modifications. This matter, as well as so many others, is described so fully by Mr. Green that the speaker need not refer to it.

The parks and the boulevards connecting the parks in Chicago are under the supervision of three commissions appointed by the State, and they are somewhat independent of the city officials. The South Park Commission has made extensive experiments with bituminous road materials, and appears to be getting very good results with a fine asphaltic concrete mixed in a specially-designed portable asphalt plant and laid by its own force. This so-called "mineral rubber" pavement, which is similar in some respects to the bitulithic pavement, is laid either on a concrete or a finished macadam foundation. It has not been down long enough to demonstrate its durability. Considerable asphaltic concrete wearing surface is also being laid by the municipal asphalt plant at Pittsburg. In Buffalo and in Rochester fine stone which will pass a screen of $\frac{3}{8}$-in. mesh and be retained on a $\frac{1}{8}$-in.

screen has been incorporated into the regular asphalt sand wearing surface, to the extent of about 20 per cent. This has been in use in Buffalo for four years, and it is proving satisfactory. With the use of fine stone the wearing surface is not so slippery, and it does not wave and creep as much as the ordinary sheet asphalt, but it requires closer attention in raking and rolling to secure a smooth uniform surface. Mr. Thomes.

In Rochester, it was stated that almost all the sheet-asphalt binder stone was coated with coal-tar and also that the wearing surface along the gutters was painted with coal-tar instead of asphaltic cement. This is not the customary practice, and it is a point which merits further discussion. The asphalt and other pavements in Rochester are in very good condition.

Considerable bituminous macadam work is being done by the various State highway departments.

There is such a diversity in the paving practice of different cities that it is difficult to compare the various pavements unless one has complete detailed information on all conditions affecting the work, including records of tests, construction, traffic, wear, etc. What would be considered heavy traffic in one city would be light traffic in another. Steps should be taken to enlist the co-operation of the various cities, as far as practicable, in adopting uniform methods and records of paving work, so that the results might be comparable and of mutual benefit to all.

RICHARD LAMB, M. AM. SOC. C. E.—Attention is called to a few important considerations in connection with that part of Mr. Green's paper which treats of wood block pavements. Mr. Lamb.

No specifications issued by engineers show less digested thought than those for wood block pavement. This, however, is quite natural, as the basis of the subject is essentially chemical and not engineering, and the promotion of the business has not been in the hands of engineers, but of manufacturers, whose aim, as a rule, seems to have been to advocate specifications which would insure a monopoly for their particular organization rather than the ultimate best interests of the wood block paving business.

Engineers have been led to specify that dead oil of tar obtained from coal only shall be used, and to specify in the next paragraph, that 25% of resin—a product of the pine tree, and not of coal—shall be put into the heated creosote oil and be injected into the wood blocks. Further, they have been led to demand that specimens of the oil, the resin, and the resin and oil mixed, be submitted with bids for their work, as though they were not aware that what they were calling for was not resin, but pitch.

It is generally known that some years ago wood block pavement treated with pitch, was laid in New York City and proved a failure. The treatment of wood with resinous vapors was tried by Lukin, at

Mr. Lamb. Woolwich, in 1812, and proved an absolute failure. The Haskin Vulcanizing process solidified the resin in heart pine, but was of little value. The plant, which was built in New York City, was used subsequently for the creo-resinate process, which combines the use of resin with the commonly accepted wood preservative, creosote oil.

Mr. Green hints that, in Chicago, wood block paving promoters have been trying to dictate the specifications. He states: "It was early recognized that to use as a standard the treatment advocated by any one company was anything but scientific."

Recently the specifications for paving in a nearby city were brought to the speaker's attention. The engineer, after specifying the creo-resinate process, demanded the submission of samples of wood blocks taken from some street in New York City in which they had been laid for five years, stating that: "It is not the intention of these specifications to permit experimenting." That engineer was an honorable man, but his error lay in not having permitted the specimen blocks to be taken from other cities where they had been in use successfully for five years, as only one company manufacturing wood blocks could meet this feature of the specification, and hence that company could control the contract. It is evident that this engineer would welcome a standard specification which would eliminate, as far as possible, the experimental features of wood block pavements. To his credit, however, he inserted a clause stating that if a bidder proposed to use blocks other than those specified, a sworn affidavit describing the materials and samples of the same, must be submitted with the bid. The bids were $3.16, $3.52, and $3.56 per sq. yd., respectively. The last two bids were submitted by companies affiliated with the manufacturers of the blocks laid in New York City five years ago. The low bid was for standard creosoted blocks.

Any of the well-established creosoting companies can put resin in their creosote oil, and they can use creosote containing the heavier tar, but engineers cannot get them to furnish blocks other than those treated with standard or regular dead oil of coal-tar, which has passed the experimental stage and has attained results showing a life of 40 years or more.

In 1880 the Society appointed a committee to report on the best preservative for wood. After a long and careful investigation, the committee, in 1885, unqualifiedly reported in favor of standard dead oil of coal-tar.

Rotting is caused by the fungi and bacteria germinated by the alternate wetting and drying of wood. If wood is kept always wet or always dry, it will not decay for a great many years. It is a fact that 12 lb. per cu. ft. of dead oil of coal-tar has preserved wood for more than 30 years. The poisonous or antiseptic nature of the naphthalene in the dead oil of coal-tar undoubtedly had much to do with this result,

as it would require more than 20 lb. per cu. ft. to saturate the pores of the wood thoroughly so as to exclude the moisture completely. Mr. Lamb.

It has been argued by good engineers that the requirements for the treatment of wood blocks for paving differ from those for wood to be used for other purposes, in that the blocks must not only be preserved from decay, but must be made water-proof in order to keep them from swelling. Experience has shown that when blocks have been properly treated with 16 lb. per cu. ft. of the dead oil of coal-tar commonly used by standard companies, they do not swell more than those treated with creosote and a special water-proofing material.

The speaker knows of an extensive pavement, where 16 lb. per cu. ft. of regular creosote oil was used, and there has not been the slightest evidence of swelling, in spite of the fact that the blocks were laid very tightly.

As Mr. Green observed, Trautwine states that white pine expands 1 part in 440 000 for each degree, Fahrenheit. With the proper number of joints, there is no trouble from expansion; therefore, in treating wood blocks, the attempt should be simply to provide the preservative which has been proven by actual experience to make the wood most durable.

The special water-proofing materials penetrate the surface of the blocks only a short distance, and are not stable. When the blocks are cut for breaking joints, the interior surfaces show none of the special water-proofing, and, if not otherwise properly treated, they make an entering place for moisture.

In 1894 the speaker made some observations on the subject of wood preservatives,* in which he contended that, in dead oil of coal-tar, naphthalene was the essential attribute for preserving the wood. He has modified his belief in this regard. Since that time the United States Forestry Department has made extensive analyses of preservatives in woods which are known to have been treated and exposed for a number of years. The results for some of the paving blocks are given in Table 3.

TABLE 3.

Average service.	Creosote left, in pounds per cubic foot.	Distillation of Extracted Oil:							
		To 205°	205° to 245°	245° to 270°	270° to 320°	320° to 420°	Residue above 420°	Solid naphthalene.	Solid anthracene.
23.6 years.....	15.70	29%	21.34%	21.39%	41.74%	11.25%	18.64%	12.52%	40.40%
9 years poor service	5.77	.....	14.41%	19.27%	27.68%	19.03%	9.93%	.....	.....

* *Transactions*, Am. Soc. C. E., Vol. XXXI, p. 238.

Mr. Lamb. This would indicate that the effective preservative is most largely naphthalene and anthracene, and not the tar acids or the heavier distillates which go over at a higher temperature than 400° cent. The latter constituents are mostly tar, and have a specific gravity of about 1.2.

The specific gravity of the dead oil of coal-tar used by standard creosoting companies is not less than 1.03, or more than 1.10. If a much heavier oil than 1.10 is used, great heat in the cylinders is necessary to liquefy it, and keep it liquefied, so that it can be injected into the wood.

The speaker has built a number of kilns for drying lumber, and has observed that when the heat is greater than 260° Fahr. the wood is cooked and made brittle, the cells are broken down, and the wood is injured. If the strength and life of the wood are to be preserved, those attributes of dead oil of coal-tar which have to be heated excessively in order to inject them into the wood, cannot be used.

Captain John C. Oakes, U. S. A., in a paper entitled "Difficulties in Practical Work of Creosoting Timber," states as follows:

"An oil having a specific gravity as a whole of more than 1.07, should be looked on with suspicion and investigated. Unless it is almost wholly lacking in the low-boiling fractions, it possibly has some hard pitch or undistilled coal-tar mixed with it."

A number of Government engineers specify that not less than 40% of the oil shall be naphthalene, and, at one time, the speaker was of this opinion. A large consignment of creosoted piles for use in the construction of a Government railroad trestle, was rejected by the Government engineer, because the oil contained 45% of naphthalene. The specifications read that not less than 40% of naphthalene should be in the oil. The engineer argued that he knew nothing about this substance, and that if his report showed 45%, the Washington authorities might blame him for accepting piles having too much naphthalene in them. The speaker laid the claims of the authorities on wood preservation before the Chief of Engineers, U. S. A., showing that naphthalene was the essential constituent of oil for preserving wood against rotting and the teredo. The Chief Engineer telegraphed to his subordinate at once to use the piles, and expressed surprise that the work had been held up for so obvious an error in the reading of the specifications.

The speaker has since concluded that this subordinate engineer was more nearly correct in his conclusions, as far as the value of the oil is concerned, than even he himself knew. If too large a percentage of naphthalene is specified, it is almost certain that a large percentage of the tar acids or lighter volatile attributes will accompany the oil. These soon leave the wood.

If the degree of heat specified for distilling is greater than the minimum necessary to extract all the lighter oils and some of the naphthalene, the volatile oils will be excluded, and if the maximum degree of heat be specified, which will fall below that necessary to take over the heavy tars, the oil will contain most of the naphthalene and nearly all, if not all, the anthracene. These degrees of heat should be positively determined, and then incorporated in specifications. Mr. Lamb.

Captain Oakes stated:

"If the creosote is obtained from works where the carbolic acid is extracted, the oil will have a low percentage of tar acids. If naphthalene has been extracted, the oil will be low in that quality, and if roofing pitch has been manufactured, there will be almost no anthracene, because the distillation ceased before the oils containing anthracene were distilled over."

From these facts it may be seen that useful information as to the probable value of any creosoting oil can be obtained in advance of an analysis, by ascertaining the kinds of materials produced by the manufacturers who are to furnish the oil.

A consideration not less important in wood block paving is the kind of wood to be used. After using many kinds of hard woods in Europe, engineers have found from experience that Baltic pine, *Pinus sylvestris*, is the best and most durable. This wood corresponds to the American *Pinus mitis*, or shoft-leaf sap pine; but it is not like the *Pinus rigida*, or old field pine, which is also short-leafed; the latter is very coarse-grained.

In the East it has become common practice of late, to call for *Pinus palustris* or long-leaf or heart pine. It is questionable whether this is best. Short-leaf pine is the stronger, although of shorter life than the long-leaf pine. However, when short-leaf pine is creosoted, it lasts as long as the long-leaf.

While the short-leaf pine has less resilience, it will stand the wear of wheels longer than long-leaf pine, on the same principle that the best bearing metals are not the hardest.

It is likely that tupelo or black gum will come into popular favor. There is a vast quantity of this wood in those districts of the United States where creosoting works are located. At present it is scarcely used for any purpose. The negroes say that black gum is so mean that even the lightning will not have anything to do with it, as it is commonly believed that lightning never strikes black gum trees. The wood is hard to split; it has little fuel value, and has a bad feature of warping and twisting. However, the speaker has manufactured a large quantity of it into long slender poles which were required to be straight and would be ruined by warping or twisting. The method was as follows: The trees were girdled when the sap was down. The sap rises twice a year, in March and in August. After letting the

Mr. Lamb. girdled tree remain for one year, it was then ready to be made into timber or lumber, and would keep its shape. It was also in better condition to creosote, and the speaker believes that the wood will wear better than any that could be obtained at the same cost.

The object of these remarks is to show, as briefly as possible, some of the considerations that make it important for the Society to appoint a committee to investigate the subject thoroughly, and to report on the best complete specifications for wood block paving.

Mr. Marr. WILLIAM W. MARR, M. AM. SOC. C. E. (by letter).—In paving along the outer rails of street-railway tracks, the author has said that the blocks should be set ½ in. above the rail, so that, after rolling, which will drive them down ¼ in., and a short period of time in which traffic will drive them down another ¼ in., they will be on a level with the top of the rail. Assuming that the concrete foundation is substantially rigid and does not settle appreciably, and also assuming that the blocks in the joints are comparatively tight, sand 1¼ in. thick, with the slight tamping which accompanies the laying of the blocks, cannot be reduced to an average thickness of ¾ in. over any considerable area, by the combined action of an ordinary steam roller and ordinary traffic covering a reasonable length of time.

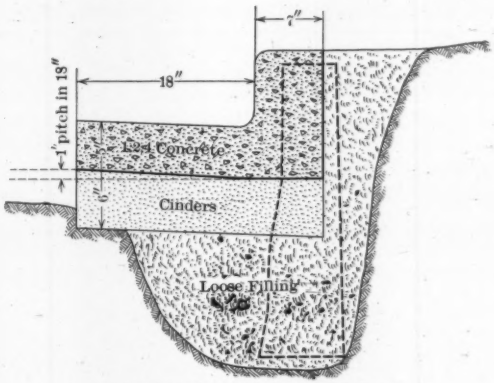
More settlement will occur along the rail than anywhere else. This is due to the fact that the sand bed is pumped from under the blocks, through the few unavoidably loose joints, by the combined action of the vibration of the rail and the water which is held, by placing the brow paving higher than the rail. The water percolates through the loose joints, and when it is forced back again by the action of traffic, it carries the sand with it, and the condition illustrated in Fig. 2 is inevitable. If, on the other hand, the pavement is laid, with a little more than the average care, to the level of the lower edge of the bevel* on the rail (about ¼ in. below the top of the rail), and then after rolling and before the top dressing is applied, such spots as have gone down next to the rail are relaid to the proper elevation, all of the pavement outside of the right of way will be below the top of the rail. This admits of proper drainage, and, in a nearly level country, is of the most vital importance.

After a season's wear repairs will be necessary, but they will be comparatively trivial, while if the pavement is laid above the rail, proper drainage, without relaying the greater part of it, will be impossible at any time during its life, and, as the author says, "The whole life of practically every pavement depends largely on this" (the drainage).

A few criticisms of methods in use which have not given complete satisfaction and notes on the subject of maintenance may be of interest.

*This bevel may be an excellent thing for facilitating better contact between the rail and wheels, but it is also an important factor in the destruction of the adjoining pavement.

Fig. 4 is a typical cross-section of the combined curb and gutter at present in use in Chicago, though the height of the curb above the gutter, 8 in., is probably a little above the average. The materials are Portland cement, fine granite screenings, and crushed granite, in the proportion of 1:2:4. The stone is required to be "clean, free from dust, loam, and dirt, crusher-run, and of varying sizes which shall pass through a ring of one and one-half inches (1½ in.) internal diameter and be held on a ring of one-quarter inch (¼ in.) internal diameter." All exposed surfaces are covered with a finishing coat of mortar, ½ in. thick, composed of Portland cement and fine granite screenings in the proportion of 2:3. Mr. Marr.



TYPICAL SECTION OF CONCRETE CURB AND GUTTER SHOWING POSITION WITH RESPECT TO EXCAVATION FOR REMOVING OLD LIMESTONE CURB.

FIG. 4.

Failure from poor workmanship, that is, from not using the required amount of cement or from using an inferior quality of cement, is very rare, and yet the whole practice of curb and gutter construction, as now carried on in Chicago, may very properly be considered a failure. To convince oneself of this fact it is only necessary to examine any street where the pavement is worn enough to be resurfaced. It will be found that only in a few cases can any of the old curb and gutter be used in repaving.

There are many reasons for this, but the principal ones are lack of proper provision against expansion (in adjacent concrete masses as well as in itself) and the instability of the cross-section.

The damaging effect of expansion in the curb and gutter itself is overcome fairly well by cutting joints at short intervals, but nothing

Mr. Marr.

has been done to offset the tendency of concrete pavements and foundations, a specific case of which was mentioned by the author, to overturn the curb and gutter. This is a serious injury, and is sufficient in itself to render the curb and gutter unfit for use in a second pavement, to say nothing of its interference with the proper drainage of the first.

A large amount of the breakage in curbs and gutters is due to the expansion of concrete sidewalks and carriage walks. Different means of preventing this have been tried with very limited success, owing to the fact that no one method seems to be very generally applicable, and, as the author has stated, the city has never had a sufficient force of technical men for thorough supervision. Some years ago an ordinance was passed prohibiting the laying of concrete walks within 1 in. of the curb. This ordinance was never rigidly enforced, and even in places where the proper space was left, it was found that the frost in winter did about as much damage as the heat in summer would otherwise have done.

Lately, the Board of Local Improvements has ordered that all small spaces between the ends of cement sidewalks and the curb be filled with the same material as that specified for the wearing surface on asphalt streets.

A large part of the paving work done in Chicago each year is in replacing the old cedar block streets with a more modern pavement. Combined curb and gutter has also taken the place, in a great measure, of limestone curbstones. The old limestone curbs are 3 ft. deep, and when about to be pulled out, a trench is dug to the bottom of the curb on its roadway side. On account of the settlement of the curb the bottom of the trench is always a little more than 3 ft. below the proper grade. These trenches are carelessly back-filled, as a rule, and never rolled, as is the case with the remainder of the roadway.

Where the roadway is not changed, the combined curb and gutter is built directly over these trenches, and usually very soon after they are back-filled. This method leaves from 12 to 24 in. of loose filling between the bottom of the 6-in. cinder bed (which the writer believes to be worse than useless), for the curb and gutter, and the solid earth beneath, and it is small wonder if settlement results.

It may well be asked, why are such conditions tolerated? The City of Chicago does not employ a sufficient number of inspectors to give this part of the work constant supervision, and relies on the guaranty of the contractor to make any repairs necessary. The contractor, while using every reasonable effort to get his men to do the work well, evidently seems to think it cheaper to make what repairs are required of him than it would be to employ more foremen. This, however, seems to the writer to be a mistake, for the maintenance requirements are becoming more and more exacting each year.

In addition to the settlement already mentioned, there is a greater

source of trouble, namely, the overturning tendency of concrete foundations, and the similar effect produced by the action of the steam roller in rolling the subgrade. Mr. Marr.

As the gutter is built with a lateral pitch of 1 in 18 on both its upper and lower surfaces, the expanding concrete foundations will tend to raise the outer edge of the gutter. A similar effect is also produced by the roller when rolling the subgrade. This is particularly noticeable in clay soil, where it frequently happens that a gutter, originally set 3 in. below the top of the curb, is pushed up until the outside edge is 2 in. above the curb.

While it is undoubtedly true that a good and permanent piece of curb and gutter of the design now used in Chicago may be put in, it is the writer's belief that the conditions under which it is practicable are rare. It also seems to him that it is folly to attempt to make a design which will be in any way general in its application.

A combined curb and gutter has many advantages over a stone curb, and, skillfully handled, can be made to "cover a multitude of sins" in the way of meeting existing permanent improvements and making a more sightly and workmanlike job; but, with a uniform cross-section (as regards the gutter), it also has serious disadvantages. The design should be governed by the kind of pavement to be used, the nature of the soil, the amount of traffic expected, the value of the surrounding property, whether it is to replace a stone curb recently removed, etc.

In order to encourage the widest distribution of traffic, the writer believes that, on streets where there are car tracks, or where there is very heavy travel, or where the pavement has a good and substantial foundation, the crown should be considerably less than would otherwise be necessary. Contractors, as a rule, like a heavy crown, because it drains the pavement quickly, and, in most cases, saves grading; but the writer is of the opinion that, except for macadam pavements, the smallest crown consistent with the proper drainage and the strength of the foundation will give the most satisfaction.

Chicago has grown so rapidly that it has been quite impossible for its municipal authorities to satisfy fully the ever-increasing demands made upon them, yet from time to time there have been some very notable achievements in the way of handling a large volume of business with few men and with great efficiency. Among the most praiseworthy of these is the manner in which the problem of the maintenance of paved streets under "reserve" (where the contractor's guaranty has not expired) has been met and solved.

In 1907 Chicago had about 600 miles of streets under reserve, embracing about 1 600 contracts by some twenty-five different contractors. No systematic efforts were made to enforce the provisions of the contracts relating to maintenance, and the various contractors did about as they pleased in this regard. All openings for laying or repairing

Mr. Marr. pipes and conduits were kept pretty well in hand by means of a record of permits, the city police force being relied on in a great measure to see that any one engaged in making an opening had a permit from the city. After an opening had been made, the contractor was notified to repair it, and was paid at the rate fixed in his contract by the person making the opening.

As there was no inspection on the repair work, other than a superficial examination after the work was done, it may be easily believed that the methods fell into an abuse that amounted to an absolute scandal. In the meantime the volume of work was increasing at such a rate that the city was absolutely powerless to cope with the situation. At this time Mr. John B. McInerney was appointed Superintendent of Maintenance and Repairs for the Board of Local Improvements, and he has been remarkably successful in organizing his department and systematizing his work.

All repair work on streets under reserve is now done under the eye of an inspector and in strict accordance with the terms of the original contract. One of the most vicious practices which has been wiped out completely is that of making an opening wider or larger at the bottom than at the top. This is no longer tolerated, and any attempt to do it is sure to bring the punishment it deserves.

Another piece of bad practice which is no longer permitted is that of crowning the paving over a trench to allow for settlement. All repairs are now made on the assumption that the paving will remain where it is placed, and failing in this it is torn out and repaved at the contractor's expense. This is made possible by holding a portion of the money due the contractor during the period for which he has given a guaranty. The old method of depending on the contractor's surety bond has proven to be a mistake, and, while a bond is still required, it is regarded as a very inadequate protection. Of the amount of the final estimate of the money due the contractor, 5% is now held and 1% is paid each year after any required repairs have been made.

In addition to the 150 000 sq. yd. of repairs of pavements made necessary by wear, there are now about 20 000 openings made each year with an average area of about 100 000 sq. yd., and it will be readily seen that the task of replacing the pavements in a proper way is not a small one. Mr. McInerney, who seems to have a genius for this line of work, believes that close supervision and good workmanship are even more necessary than the best materials in getting the most satisfactory repairs, and the result of this has been that Chicago now has really "invisible patches."

One of the most perplexing problems now confronting the municipal engineer in Chicago is the drainage of the business streets. This arises as a result of forcing indiscriminately the various public service corporations to put their pipes, conduit wires, etc., underground. The

city has required that all telephone, telegraph, and electric power wires and cables be put underground, but it has not designated or allotted any particular place for them. As a consequence, the companies, acting under an order of Council, and with a permit from the Department of Public Works, have proceeded to pick out, each one for itself, the best place it can find and do its work with less regard for how the city's interests are conserved than the city might reasonably desire. It happens frequently that when an attempt is made to build new catch-basins or to make sewer connections, there is no room in the roadway above the sewer to do so. That some means of controlling the space occupied by the public service corporations' properties be adopted, is absolutely imperative. Mr. Marr.

NELSON P. LEWIS, M. AM. SOC. C. E.—A paper describing the paving practice of a great city like Chicago is naturally of much value to the Engineering Profession, especially to those engaged in municipal work, even though it may contain little that is new. The author has covered several points which the speaker believes should be emphasized or elaborated, and this discussion will relate almost wholly to those portions of the paper which refer to the use of wood block pavements. Mr. Lewis.

The author calls attention to the danger of using blocks the depth of which is the same as their thickness, for the reason that with a creosoted block, it is sometimes difficult to tell whether the wood fiber is placed horizontally or vertically. In the speaker's opinion, this danger is not sufficiently emphasized, and he believes that where the width and depth of the blocks are the same, a considerable percentage is improperly laid. Nor does he believe that the author's suggestion that this difficulty would be remedied by a difference of $\frac{1}{4}$ in. in these dimensions goes far enough, but is of the opinion that the difference should be not less than $\frac{1}{2}$ in.

The author refers to European practice in the laying of wood block pavements, and, in view of the fact that the life of such pavements is considerably less in European than in American cities, he concludes that the practice in this country is the better. The speaker believes that this conclusion is warranted, but the reason for the difference in the wearing value of the pavements should be understood. It would probably be fair to take the City of Paris as typical. In that city the wood which is used in paving is considerably softer than the yellow pine in common use in the United States. Further, the treatment of the blocks is very different, being very superficial and consisting of little more than the immersion of the blocks in creosote, the penetration obtained in this way amounting to only several millimeters. Under the prevailing American practice, the blocks are thoroughly impregnated with oil. French engineers estimate that the cost of thorough impregnation would amount to 60 cents per sq. m., while the simple steeping of the blocks costs only 8 or 9 cents per sq. m., and it is be-

Mr. Lewis.

lieved that inasmuch as the blocks will probably wear out before they decay, the additional expense of thorough impregnation is not warranted. In several instances the author refers to translations of papers by Messrs. Tur and Mazerolle, both of whom are connected with the municipal service of the City of Paris, and states that the latter notes "with approval that wood blocks, creosoted under pressure, are still in good condition after a service of fifteen years." He doubtless refers to two instances given by M. Mazerolle, in one of which Swedish pine blocks creosoted under pressure have been in use for 14 years and are still in place, while their wear is insignificant, but it is noted that the locality is little frequented by carriages. The other is that of streets, in the vicinity of a school, which are little frequented by traffic and are paved with creosoted Norwegian pine, the pavement having been in use for 13 years. It is quite evident that the theory of the French engineers is that, where the wood is not sufficiently hard and durable to resist traffic for a number of years exceeding its life when treated in the manner prevailing in that country, it is useless to resort to more costly methods of treatment.

The experience of the City of Paris in the use of hard woods appears to justify the engineers in charge of the highway service in the belief that they are inferior to the soft woods in common use. M. Mazerolle gives the average life of pine paving blocks as 7 years and 7 months, while the average life of a number of the hard woods which have been tried has been as follows:

Karri, 5 years and 4 months,
Jarrah, 6 years and 4 months,
Teak-wood, 6 years and 4 months,
Lime, 5 years and 9 months,
Ironwood, 5 years and 5 months.

The theory is that these exceedingly hard woods, having little elasticity, transmit the shock of the traffic directly to the concrete foundation, which is speedily injured; and it is noted that, while these hardwood pavements remain in good condition for two or three years after construction, they then quickly deteriorate, and the period which separates the time when the conditions cease to be entirely satisfactory from the time when they begin to be particularly bad, is quite short. The explanation of this is that when the joints begin to open and the blocks become loose, each one forms a hammer which transmits to the underlying concrete any impact due to the passing of vehicles, and the concrete is soon pounded to pieces. The theory of French engineers is that the concrete foundation is the real pavement which sustains the street traffic, and that wood or asphalt, or even stone blocks, form simply a wearing surface through which the load is transmitted to the underlying foundation, and that the function of the wood or asphalt

is to protect this foundation, and, at the same time, afford a wearing surface which is both smooth and elastic. Mr. Lewis.

The source of supply of the paving blocks generally used in France is the forests of the Department of the Landes, which forests had a somewhat peculiar origin. They are located along the coast on what were formerly sand barrens, the fine sand from which, being blown inland, was covering and destroying valuable agricultural land. Efforts were made to check these sand storms by some vegetation which would thrive, and after several unsuccessful attempts, hardy pines were planted, which, in the course of many years, have developed into the forests which are now furnishing almost the entire supply of paving blocks for the French capital.

The author refers to criticisms of the practice in New York City of laying blocks only 3½ in. in depth, and, further, says that in a report to which he refers "New York City has been criticized severely by various engineers for using such shallow blocks." This, he maintains, is not good reasoning, as it is not probable that a granite pavement ever failed because of the shallowness of its blocks. The speaker, who is responsible for the report referred to, cannot recall anything in that report which would justify this statement, although it was noted that both wood and granite blocks of exceptional depth are used in Great Britain. In fact, it is the speaker's opinion, based on observation, that the depth of granite blocks, as commonly used in the United States, could be substantially reduced, especially if the blocks could be cut more carefully, with uniform depth and smooth surfaces. Small cubes cut in such a manner have given most satisfactory results in English cities.

P. E. GREEN, ASSOC. M. AM. SOC. C. E. (by letter).—Some good points, and a few inquiries, are brought out in the discussion of this paper. Mr. Crosby believes that a minimum limit of less than 20% loss for the rattler test of brick paving blocks should be specified, in order that a lack of uniformity may be avoided. It is the writer's opinion that this is not necessary, for the reason that the rattler test is a very severe one, and is made with a number of bricks at one time and no one brick is taken as the criterion, but the average loss of the entire set is measured, and there are very few charges but approach very nearly to the allowable loss. Mr. Green.

With regard to stone blocks, it is his suggestion that possibly a similar test might be desirable to determine the wear and likelihood of becoming rounded and slippery under traffic. Again, it is the writer's belief that this is not necessary, for it is very easy to detect by inspection the character of a granite block, as those that are very slippery generally show it.

It has also been suggested that, possibly on account of the limitations of the number of quarries from which blocks are allowed to be used, the price might be increased. This has not been found to be a

Mr. Green. fact, as the quarries that were condemned are those that furnished stone not at all fit for the purpose, and the number of quarries that furnish good stone is sufficient and scattered far enough apart to create real competition.

In regard to concrete pavement, the writer is not able to give any figures as to the cost of repairs, as they are not available. He agrees with Mr. Crosby that they wear unevenly and that the problem of keeping such pavements in repair is extremely difficult.

Mr. Thomes has suggested that stone curb could be set in concrete made with small stone, but thinks that 24 in. is too deep for this curb. The experience in Chicago is that 24 in. is not deep enough, for the reason that the roller, in rolling the subgrade, disturbs the alignment. It can readily be seen that this is a necessary accompaniment of a shallow curb. Taking a granite street as an example, and assuming that 7 in. of curb is exposed, that the granite block is 5 in. deep, the sand 2 in. deep, and the concrete 6 in., this would leave the curb exposed above the subgrade a distance of 20 in. A roller, in this case, would undoubtedly disturb the alignment very seriously.

Mr. Thomes' objection, however, to the wearing surface of a concrete gutter is a very pertinent one, and is being vigorously debated by the City Engineers of Chicago at the present time. It is the general opinion that the gutter should be narrower than the present standard (18 in.) and also deeper, and have a heavier and more resistant wearing surface. One of the difficulties to be encountered is the fact that Chicago is very flat, and the drainage must be made entirely by varying the depth of the gutter below the curb. Hence, as a rule, the water does not flow very rapidly over the entire street, and on an asphalt street a slight obstruction holding the water will cause the asphalt to rot. It is the general opinion of the engineers engaged in this work that the gutter should be narrowed, and several designs have been submitted for approval in which the gutter varies in width from 6 to 12 in.

In regard to the common Western practice of laying the wood blocks at an angle of from 20° to 45° with the gutter, which practice is condemned in the paper, it appears that if there are car tracks on the street and the space between the tracks is paved with creosoted blocks, considerable care must be taken with the joints or they will wear very badly under traffic when they are perpendicular to the line of the street. This is due to the fact that about every 6 ft. the rails are joined by flat iron tie-rods. The blocks are then placed rather loosely between these tie-rods and the result is that the joints are badly battered, especially at the junction of the blocks and the tie-rods. This fault may be remedied by using special blocks, cut so as to fit over these tie-rods, the whole pavement then being tightened up with wedges.

Mr. Lamb evidently misunderstood the writer and came to the con

clusion that the contractors had tried to dictate specifications in Chicago. When the blocks were first used there was certainly lack of knowledge among the engineers as to the proper kind of treatment, etc., and it was but natural that they should use specifications prepared by the promoting companies. It is but just to say, however, that since the city's laboratories have been investigating the properties of the various preserving oils, the different companies have given every facility in their power to help the engineers to a proper understanding of the problem. Mr. Green.

After a season of experience with blocks made $\frac{1}{4}$ in. smaller in one dimension than the other, the writer is of the opinion that Mr. Lewis' suggestion that the difference should be greater than $\frac{1}{4}$ in. and not less than $\frac{1}{2}$ in. is good. The problem is not that the difference between the $3\frac{3}{4}$ and 4 in. is not readily apparent, but the fact that in sawing the blocks not enough care is taken to make them exact, and there is likely to be a slight difference in the dimensions, $3\frac{3}{4}$ in. being likely to become $3\frac{7}{8}$ in., and 4 in. is also likely to become $3\frac{7}{8}$ in., and then the problem is the same as it was before the different dimensions were specified.

Mr. Marr has shown a typical section of a concrete curb and gutter, and has very properly condemned the practice of pulling the old curb and building the new curb and gutter on the filled cavity. It would seem to the writer as if the fault could be largely eliminated by simply breaking off the old curb stone and building the new curb and gutter on the old broken curb stone. This would be simpler than pulling, and would give a very sound foundation under the new construction.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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TRANSACTIONS

Paper No. 1132

THE TRACTION OF FREIGHT TRAINS AT DIFFERENT SPEEDS.

By Clinton S. Bissell, M. Am. Soc. C. E.

With Discussion by Messrs. A. C. Dennis, and Clinton S. Bissell.

The problem of railroad traction is to determine what a given tractive power can accomplish in overcoming a number of resistances, the principal of which is grade. The "ruling grade" is usually the maximum grade on a tangent, and the increase in resistance which would be introduced by curves on the ruling grade is, in the present day, done away with by equating or "slackening grade" throughout the length of the curve, with the purpose of producing a perfectly uniform resistance over tangents and curves alike. In the case of curves not thus equated, the additional resistance due to the curves must be applied as an increase to the ruling gradient throughout the length of each curve, and, if the proportion of curved line be considerable, the judgment of the engineer must decide upon what the ultimate "resisting gradient" will be.

Curves are commonly equated at rates varying between 0.02% and 0.10% of gradient per degree of curve, and 0.05% is a rate largely used for freight tracks where the speed is slow. It is well known, however, that curve resistance does not vary uniformly with the degree of curve. Among authorities on the subject, the writer finds a general consensus of opinion to the effect that 0.05% for light curves and

0.02% or 0.03% for very sharp curves is about right, with intermediate values approximately proportional. Using this as a standard, it is apparent that the amount by which the rate of the ruling grade should be reduced on curves is given by the following equation, in which C is the percentage of reduction and D is the degree of curve:

$$C = \frac{D}{D + 18} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

This reduction is for curves on ruling grades where the train is not required to stop. Using Equation 1, a 1% grade becomes 0.9% when equated for a 2° curve, and 0.47% when equated for a 20° curve. In what follows, the term, g, means either the "resisting gradient" or the "ruling grade compensated for curvature," at about this rate.

In a paper* recently presented to the Society, the writer developed the following equation, expressing the form of variation in the case of the weights of heavy freight trains moving at the slow speed of from 10 to 7 miles per hour:

$$\text{Tons of train weight behind tender} = \frac{P - 20\,g\,m}{\frac{a}{b + W} + 20\,g} \quad \ldots\ldots\ldots (2)$$

in which P = the drawbar pull, in pounds, m = the weight of the locomotive and tender, in tons, a and b are constants, W is the average weight per car of the train, in tons, and g is the rate of the ruling grade, in percentage, the ton being of 2 000 lb. For a locomotive, weighing, with its tender, 168 tons, exerting a drawbar pull of 32 100 lb., and hauling a train of modern high-efficiency cars, which is representative of ordinary conditions, this equation becomes:

$$\text{Tons of train weight behind tender} = \frac{32\,100 - 3\,360\,g}{\frac{249.6}{11.2 + W} + 20\,g}.$$

In order to adapt Equation 2 to speeds greater than 10 miles per hour, two changes must be made. As the speed increases, the drawbar pull, P, decreases; and, to a much less extent, the car resistance, $\frac{a}{b + W}$, increases. The variation in the drawbar pull is unquestionably in the form of a curve, and elaborate modern tests† have shown that the rate of change is comparatively low at high speeds, becoming

* "The Maximum Weights of Slow Freight Trains," by C. S. Bissell, *Transactions*, Am. Soc. C. E., Vol. LXIV, p. 303.

† For example, "Locomotive Tests and Exhibits, Pennsylvania Railroad System, Louisiana Purchase Exposition," 1904.

more and more rapid as the speed decreases, until near the limiting maximum value of the tractive power. At this point the curve usually exhibits a quick decline, producing contrary flexure; and, for modern heavy locomotives, this takes place at a speed of between 10 and 6 miles per hour. Such a variation cannot be expressed algebraically without great complication, but fortunately the major part of the curve, above 10 miles per hour, may be expressed very simply as an algebraic equation. From the records of the Pennsylvania Railroad Company the writer obtained the following data pertaining to a heavy freight locomotive of the type used in this study:

Speed, S, in miles per hour.	Drawbar pull, P, in thousands of pounds.
10	32.10
16	22.37
30	10.35

Observing that a suitable form of equation for a curve through these points is:

$$P = \frac{h - k\,S}{n + S},$$

in which h, k, and n are constants, and S is the speed; and deducing the constants, the expression for the drawbar pull becomes:

$$P = \frac{863.5 - 14.1\,S}{12.5 + S},$$

in which P is in thousands of pounds, and decreases as the speed increases, in proper proportion, as shown on Fig. 1.

The second step is to find an expression for the increase of train resistance, in pounds per ton, in proportion to the increase of speed. For this purpose the writer refers to the paper* "Virtual Grades for Freight Trains" by A. C. Dennis, M. Am. Soc. C. E. Fig. 1 of that paper exhibits several curves of train resistance, and, commenting upon them, Mr. Dennis says:

> The train resistance, in pounds per ton, compensated for change in velocity head, grade and curvature, was plotted for very many points and speeds for each train, and the mean of these points taken, as nearly as possible, for each train. Fig. 1 shows what is believed to be the correct resistance under the conditions given, and differs radically from the results obtained by Wellington and other authorities experimenting with light trains, cars and rails."

* *Transactions*, Am. Soc. C. E., Vol. L, p. 1.

For this investigation the writer selected the curve marked "52 Loaded Box Cars. Tare 37% of Gross Weight. Track Soft," and used only that portion which is for a speed greater than 10 miles per hour. Taking the ordinates by scale, the following was obtained:

Speed, S, in miles per hour.	Resistance, Y, in pounds per ton.
15	4.50
25	4.73
35	5.00

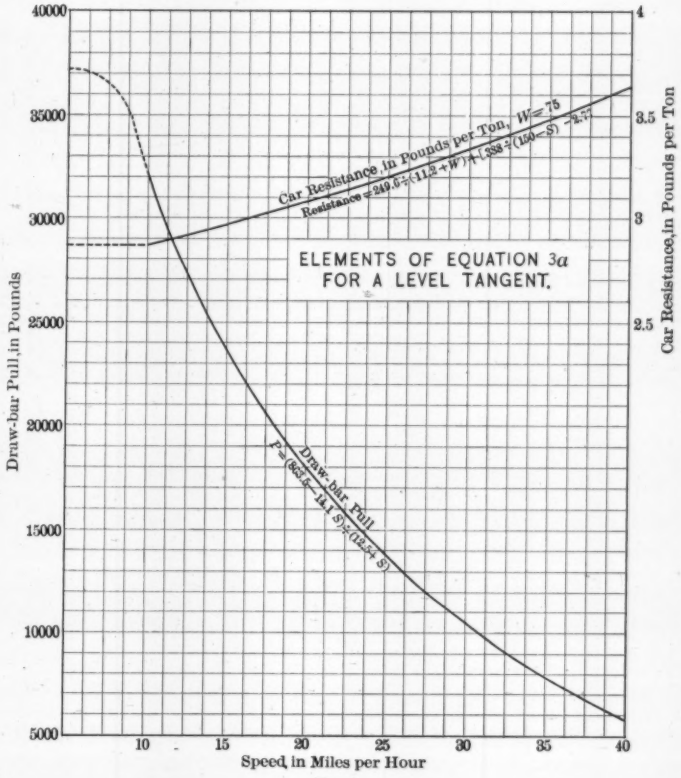


FIG. 1.

in which the resistance, 4.73, is about 0.1 greater than as shown by the curve. In the light of Mr. Dennis' comments, this slight modification will not vitiate the accuracy of the following expression for resistance,

which was deduced in the same manner as that for drawbar pull. The curve,

$$Y = \frac{388}{150 - S} + 1.625,$$

passes through the three points tabulated above. Had the modification not been made, the numerator of the fraction would have been smaller and the denominator, $150 - S$, would have become about $47 - S$. For speeds near 47 the variation would then have become much too great, since the curve has a rapid flexure on approaching its asymptote, $S = 47$. Under the circumstances, this slight change seems preferable to the complication necessary for a more exact treatment of the case.

The value of Y for a speed of 10 miles per hour, according to this equation, is 4.396 lb., and the increase in resistance at this speed must be zero, hence the constant, +1.625, becomes —2.77, and the expression for increase in resistance due to speeds exceeding 10 miles per hour is:

$$\text{Increase in car resistance} = \frac{388}{150 - S} - 2.77.$$

Denoting the constants by the letters, r, t, and c, and substituting the two expressions in Equation 2, the general equation is formed:

$$\text{Train weight, in tons, behind tender} = \frac{\dfrac{1\,000\,(h - k\,S)}{n + S} - 20\,g\,m}{\dfrac{a}{b + W} + \dfrac{r}{t - S} - c + 20\,g} \quad \ldots(3)$$

in which S is the speed, in miles per hour, W is the unit of train weight behind the tender, expressed as the average weight per car, in tons, and the remaining letters are constants, as heretofore designated. Substituting in Equation 3 the constants for the locomotive and cars used in the given example, there results the equation:

$$\text{Train weight, in tons, behind tender} = \frac{\dfrac{1\,000\,(863.5 - 14.1\,S)}{12.5 + S} - 3\,360\,g}{\dfrac{249.6}{11.2 + W} + \left(\dfrac{388}{150 - S} - 2.77\right) + 20\,g} \quad \ldots(3a)$$

It will be noted here that the train weight varies with a multiple of the square of the speed, which is in accordance with past experience.

As a matter of suggestion, the writer wishes to call attention again to the expression for increase in train resistance due to speed. Regarding the curve passing through the points, $S = 15$, $Y = 4.5$, and $S = 35$, $Y = 5.0$, as a circle tangent to the direction of abscissas at $S = 0$, and applying the principle that "the square of any distance

divided by twice the radius will equal the distance from tangent to curve very nearly"* the radius is found to be 1 000. This gives the rate of increase in resistance as 0.0005 S^2, increasing from $S = 0$. For $S = 10$ the value is 0.05, and therefore the increase in resistance

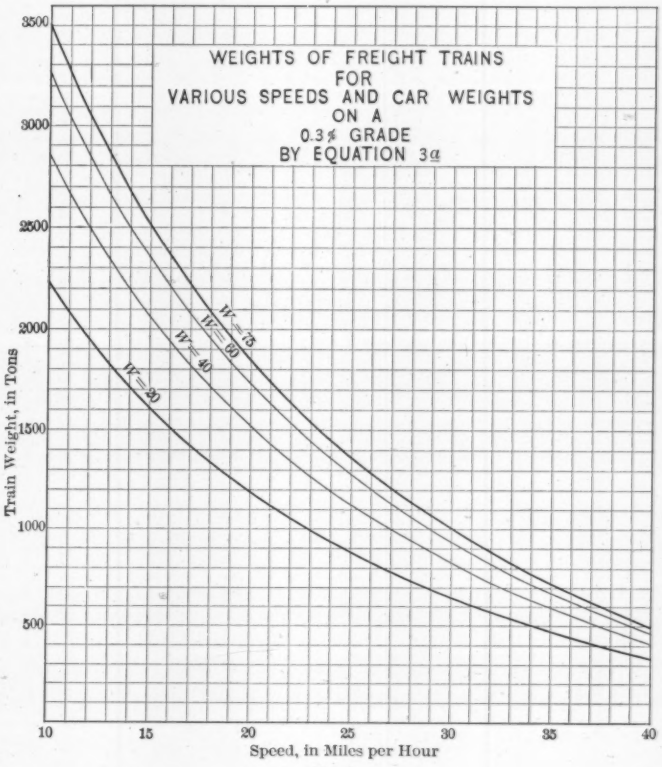


FIG. 2.

above 10 miles per hour will be (0.0005 S^2 — 0.05), which may be substituted in Equation 3*a* for the expression $\left(\frac{388}{150 - S} - 2.77\right)$, if desired. The results are practically identical, the difference at 50 miles per hour being only 0.09 lb. per ton. The writer prefers the

*This statement is to be found appended to tables designed by George T. Keith, M. Am. Soc. C. E.

expression containing only the first power of S, because of its greater simplicity, and its flexibility in adaptation to data obtained by experimental tests. The curve on Fig. 1 marked "Car Resistance" shows the form of variation for an average car weight of 75 tons.

The results of Equation 3*a* for car weights of 20, 40, 60, and 75 tons, on a 0.3% grade, are exhibited in Fig. 2; and for similar car weights on a 1% grade in Fig. 3. The great effect which the car weight has is apparent from these curves.

The writer had completed the form of Equation 3*a* before he realized that the expression for car resistance was a form over which there has been much discussion and disagreement in years past. Those interested will find in "Economics of Railroad Construction," by Walter Loring Webb, M. Am. Soc. C. E., a carefully drawn comparison of formulas for "train resistance" identified with the names of Baldwin's Locomotive Works, Wellington, Barnes, Aspinwall, and Searles. All these formulas are of the form:

Resistance, in pounds per ton = constant + multiple of velocity.

Professor Webb calls attention to the fact that only Searles' formula and Wellington's (which was based on Searles') contain a factor varying with the train weight, and he emphasizes the necessity of this element in any expression which aims to comprehend the entire range of variation. It is noticeable that the constant in the above form was evidently chosen to represent an average value for the expression, $\frac{a}{b + W}$, in Equation 3, and that, in the two formulas just mentioned, a part of the resistance is made to vary with the total weight of the train, thus rendering them more accurate for extremes of velocity.

It will be observed here that the factor, W, the average weight per car, is representative and comprehensive of all such terms as "weight of train," multiples of "length of train," "number of cars," or similar factors.

The remarks* of the late A. M. Wellington, M. Am. Soc. C. E., whose formulas are mentioned above, indicate a doubt as to his own determination of the proper total value of the various resistances. As a heading for some tabulated values he writes:

"Table 166. Train Resistance on a Level as Affected by Velocity Giving what may be considered as the ordinary working maximum, as

* "The Economic Theory of Railway Location," by A. M. Wellington, M. Am. Soc. C. E.

computed from the general formula of Wm. H. Searles, coinciding closely with the apparent indications of the most recent* tests, but possibly as much as *one-third too high* for the resistances at high speeds under favorable conditions * * *."

The principal resistances here referred to are evidently those of the atmosphere. To the writer's mind, the carefully executed tests made by Mr. Dennis have demonstrated that normal atmospheric resistances

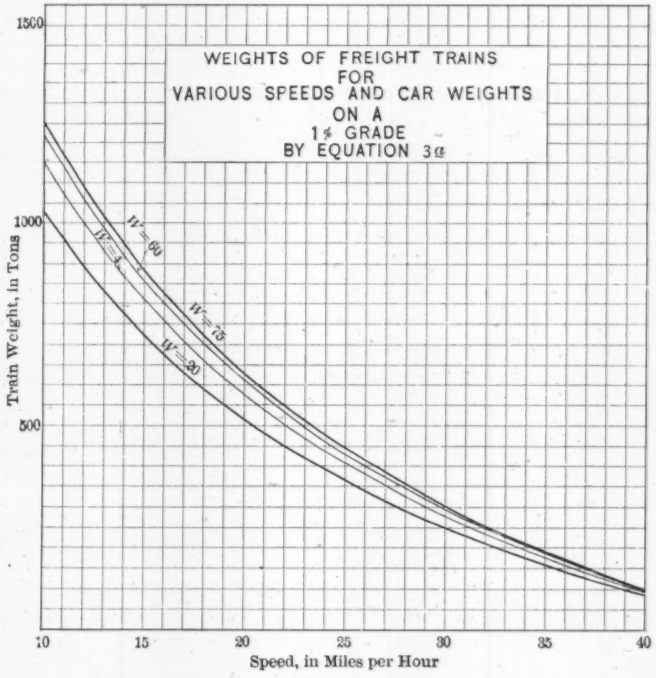


FIG. 8.

are not nearly as great as they were formerly believed to be; and, in using these tests for the development of Equation 3*a*, the writer believes that the atmospheric resistances are represented by the slight upward trend of the curve used to show this variation.

Professor Searles says:† "A formula which shall express the resist-

* Not later than 1887. (The writer.)

† "Field Engineering," by William H. Searles, M. Am. Soc. C. E.

ance of a train to uniform motion must include at least the velocity and the weight of the train and engine."

In the development of Equation 3*a*, the writer has regarded the engine as merely the extraneous agent which supplies the drawbar pull. Since the drawbar pull contains no element of the internal or other resistances of the engine, the form of development has made it unnecessary that the engine weight should appear in the equation.

It has been difficult to obtain data with which to compare the results given by Equation 3*a*, but such comparisons as could be made have been satisfactory. In formulating the equation, the great object has been to deduce expressions which would represent the variation accurately to a point well beyond the extreme limit of speed. Recognizing that the problem is not one of absolute precision, the writer has in all cases used curves involving only the first power of the independent variable in preference to more complicated forms. The constants for each of the three curves, that is, the drawbar pull, the car resistance, and the increase in car resistance, must be determined by actual tests, and, therefore, upon the excellence of the tests depends the accuracy of the results. From good tests the constants can be easily deduced for each curve by a simple algebraic process. When this has been done, the car resistance constants may be retained, and only those in the expression for drawbar pull will need to be changed in order to suit the type of locomotive used.

The great disparity between the results of formulas which have heretofore been deduced to express the resistance of a train at different speeds seems to have arisen from either a failure to recognize the effect of the relation of the train weight to the number of cars, or else from an attempt to deduce an average value to represent it, which is obviously impossible. The absolutely correct formula would require an expression for the speed and also one for the weight and resistance of each particular car, which is impracticable. The nearest approach to this form would be an equation containing the speed factor, the number of cars, and the weight of the train, and this is the form of Equation 3, except that the two latter factors have been merged into one, "the average weight per car" of the train, which is a most convenient expression for the purpose to be served.

Equation 3 is adapted for use in all cases of hauls made by locomotives of either steam, electric, or other motive power; and also in

cases of multiple-unit trains, since each unit may be considered as a separate train. The general method of developing the three curves will be the same as exemplified above, except that the form of one or more of them might need to be changed to suit special forms of cars. It does not appear probable, however, that such a change will be necessary, for many years at least, except perhaps in the curve for the increase in car resistance due to increase in speed. It is with great interest that the writer presents this development, in the hope that the discussion may bring out whatever fallacies or errors exist, in order that they may be rectified, if possible.

DISCUSSION

Mr. Dennis. A. C. DENNIS, M. AM. SOC. C. E. (by letter).—The following formula was determined by the writer after considerable experimenting with freight trains consisting of box cars of variable loading:

$$R = 2.6\ W + 85\ N,$$

in which R = Resistance, in pounds, for box cars on a level tangent.

W = Gross weight, in short tons, of the train behind the tender.

N = Number of cars in train.

Both factors vary with the class of track, rolling stock, weather, and many other minor influences, but have remained remarkably close for all test trains run by the writer.

This resistance may be considered as doubled for starting, but constant for all practical purposes after a train is in motion. Speed, wind, and temperature have little effect. The increased journal resistance which occurs when a train stands for some time is very greatly increased by extreme cold.

It is believed that ultimately freight train resistance formulas for practical rating will take the above simple form, the coefficients applicable being determined by experiment.

Mr. Bissell. CLINTON S. BISSELL, M. AM. SOC. C. E. (by letter).—The equation given by Mr. Dennis for the resistance of box cars on a level tangent, $R = 2.6\ W + 85\ N$, represents the total values of resistance and weight for an entire train behind the tender. If, for example, the weight were 800 tons, and there were 20 cars, the resistance, R, would be 3 780 lb. Dividing this by 800 gives 4.725 lb. per ton of train weight for the average weight per car of 40 tons. It will be seen that if 40 cars, each weighing 40 tons, constitute the train, the resistance in pounds per ton is the same, namely, 4.725 lb. per ton for a 40-ton car. Hence, if we change the notation to conform to the writer's,

$$R = \frac{2.6\ WN + 85\ N}{WN}$$

or, since N cancels,

$$R = \frac{2.6\ W + 85}{W}$$

which is evidently the same as $R = \frac{85}{W} + 2.6$ ..... (*a*)

This curve gives the following values:

W = average car weight, in tons.	R = resistance per ton of car weight, in pounds.
75	3.733
60	4.017
40	4.725
20	6.850

Mr. Bissell.

It will be seen that, for a 45-ton car, the value by Mr. Dennis' equation is practically the same as by the writer's form $R = \frac{249.6}{11.2 + W}$; for lighter car weights, his values are less, and for heavier cars, greater than the writer's. In order to use his values, the expression, (*a*), must be substituted in Equation 3*a* for the expression, $\frac{249.6}{11.2 + W}$.

From the appearance of the data which have been contributed on car resistance, it seems probable that a fair average will be represented by a curve passing through the three points:

W (tons)	R (pounds, per ton)
72	3.0
40	4.1
20	6.5

Nearly enough for practical purposes, the curve is $R = \frac{96}{W} + 1.7$, of the form given by Mr. Dennis, with the notation changed to the writer's form. If this expression be substituted in Equation 3*a* for $\frac{249.6}{11.2 + W}$, the resulting train weight for 40-ton cars will be nearly 20% greater than the writer's value.

However, the object of the investigation was to produce a form of variation, and not to attempt to establish constants. The constants used in illustration are conservative in value, but from what has been said by those who have commented on the subjects treated in this and in the writer's paper, to which reference has been made, there seem to have been no objections raised in regard to the forms of the various elements in Equation 3. The writer believes this equation to be true in form, rationally consistent, and particularly adapted for practical use, because of the simplicity of its parts and the consequent flexibility of the elementary forms of which it is composed. The equation is applicable only to trains at a constant speed, whether that speed be 10 miles per hour or greater. It does not apply in cases where momentum is relied on to overcome any one or part of the resistances, nor has any attempt been made to deal with the complications arising from inertia or momentum when the speed is changed. This circumstance limits its use to long hauls made on ruling grades.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1133

THE USE AND CARE OF THE CURRENT METER, AS PRACTICED BY THE UNITED STATES GEOLOGICAL SURVEY.*

By John C. Hoyt, M. Am. Soc. C. E.

With Discussion by Messrs. Arthur H. Diamant, E. C. Murphy, H. K. Barrows, C. E. Grunsky, E. Kuichling, Gerard H. Matthes, D. G. Thomas, Walter Pearl, C. M. Allen, and John C. Hoyt.

Introduction.

A current meter for measuring the velocity of flowing water comprises two essential parts: (*a*) a wheel arranged so that when suspended in flowing water the pressure of the water against it causes it to revolve; (*b*) a device for recording or indicating the number of revolutions of this wheel. The relation between the velocity of the moving water and the revolutions of the wheel is determined by rating each meter.

The earliest type of meter was the float wheel, which was used by Borda and Dupuit in the latter part of the eighteenth century, and was practicable only for measuring velocities at the surface. About 1790, Woltmann modified this wheel so that it could be used beneath the surface, the number of revolutions being recorded by a gear mechanism, which was started and stopped at the beginning and end

* Presented at the meeting of September 15th, 1909.

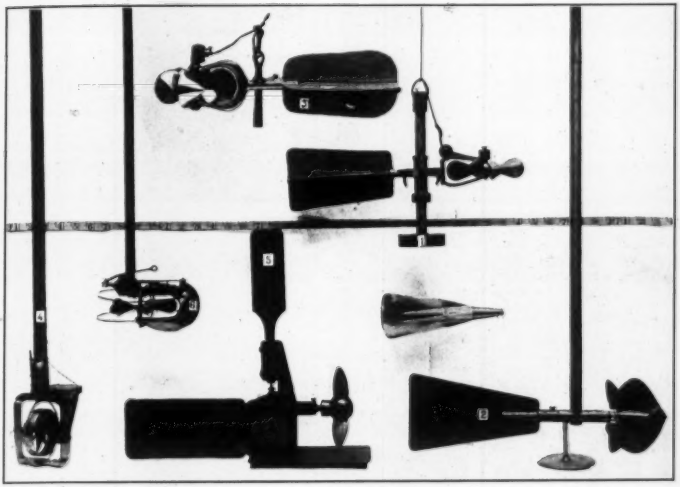


FIG. 1.—TYPES OF METERS EXPERIMENTED WITH BY U. S. GEOLOGICAL SURVEY.

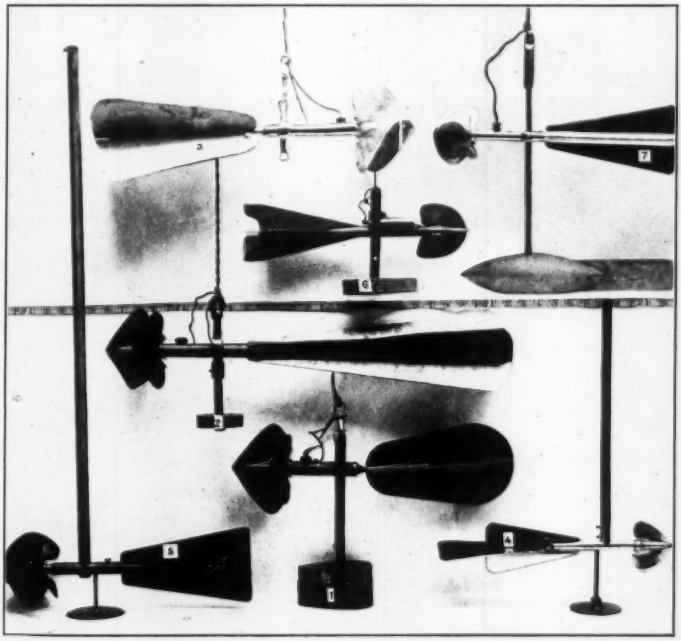


FIG. 2.—VARIOUS FORMS OF THE HASKELL METER.

of a run by a catch operated by a cord. It was necessary, however, to lift the meter out of the water in order to read it. LaPointe arranged the recording apparatus above the surface by connecting the axle with a vertical rod and beveled gear; and Baumgarten, Saxton, Brewster, Laignel, and others made various modifications of the instrument. Prior to the invention of the electric recorder for indicating the number of revolutions of the wheel, users of the meter always experienced difficulty caused by the resistance due to the recording mechanism and its tendency to get out of order.

In America current meters were earliest used in connection with the investigations of the Mississippi, started in 1850 by Humphreys and Abbot,* when the ship's log and the Saxton meter were used to a small extent and with little success.

About 1860, the late D. Farrand Henry, M. Am. Soc. C. E., Assistant on the United States Lake Survey, invented for use with the current meter an electrical recorder,† which eliminated the frictional and the other difficulties of the mechanical recorder, and made feasible the further development of the meter.

The first extended and successful series of measurements with the current meter in the United States was made on the Connecticut River by the late T. G. Ellis, M. Am. Soc. C. E., in connection with studies begun in 1871.‡ General Ellis started his work with the Woltmann meter, equipped with an electrical recording device, but later used an electrical recording meter devised by himself. The results obtained by these measurements have had an important effect on the development of stream-gauging instruments.

The earliest American patents for current meters were taken out in 1851. There are now on file in the Patent Office, classified under ship's logs, more than fifty patents for devices for measuring the velocity of water, and many other devices have been constructed but not patented. The only ones which have had much general use, however, are the Price, Haskell, Fteley, and Ellis meters (Plates II, III, and IV, and Fig. 1).

Each of the various meters has been developed to meet the requirements of some special condition, and, until recently, the use of all

* "Report upon the Physics and Hydraulics of the Mississippi River," 1861.

† *Journal* of the Franklin Institute, Vol. XCII, 1871.

‡ Report, Chief of Engineers, U. S. Army, 1878, Part I.

has been confined to special hydrographic investigations in connection with some public work, municipal, State, or Federal. The present widespread interest in hydraulic development has created such a demand for measurements of the flow of streams that the current meter is now in general use, and is essential to the equipment of the engineer engaged in hydraulics.

In 1888 the United States Geological Survey began the gauging of streams. The plans called for measurements in all sections of the country, and the conditions to be provided for presented an infinite variety in combinations of range in depth, width, and velocity. No adequate meter was available for work of this extended nature, and, furthermore, elaborate equipment and methods were prohibited by limited funds. It was necessary to devise or adapt an instrument which could be readily carried in the field and operated by one man, either from a bridge, a boat, a cable and car, or by wading.

After experimenting with various types (Plates II and III), the engineers of the Survey developed a meter combining the essential features of the Price acoustic and large Price electric meter (Plate III). This is known as the small Price meter, and has been in general use in the Survey work. Modifications in its construction have been made from time to time, based on the experience of various engineers, until now it represents the experience of more than twenty years of stream gauging (Plate IV).

The methods developed by the Survey engineers are believed to represent the best practice in this line of work. The Survey's data are now used extensively in hydraulic development in all parts of the country, and its methods have been adopted in similar work by many engineers in all parts of the world. A description of the instruments and their use, therefore, should be of interest to all engineers.

General Features of Current Meters.

Current meters, in general, may be divided into two classes: direct action and differential action, the division depending on whether the water, in revolving the wheel, does or does not exert a force which tends to retard the motion of the wheel.

The wheels of the direct-action meters consist of flat or warped-surface vanes, set on a horizontal axis, which revolve by the direct pressure of the water against the blades. In a meter of this type the

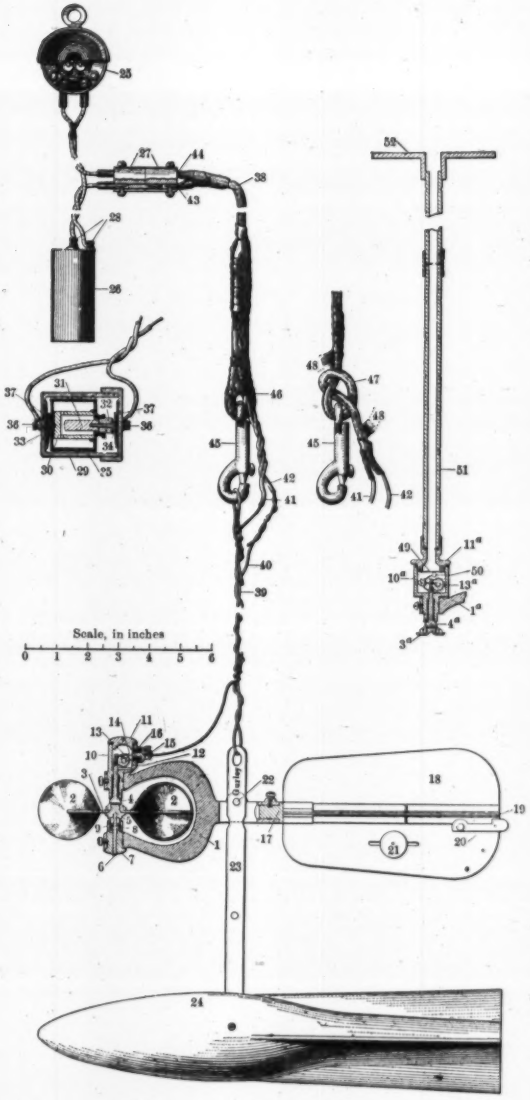


FIG. 1.

friction increases as the velocity decreases. The revolutions of the wheel are quite rapid for high velocities, but the resistance due to the area of the cross-section of the meter against the water is small. The principal direct-action meters are the Haskell and the Fteley (Plate II).

The wheels of the differential meters are made up of a series of cups which revolve on a vertical axis. The pressure of the water on the cups, being less on the convex than on the concave side, causes the wheels to revolve. In a meter of this type, the friction increases with the velocity, but the increased frictional effect is overcome by the increased motive power due to the velocity. The area of the cross-section offers a larger resistance to the current than in the direct-action meter, but the wheel revolves more slowly owing to the retarding motion caused by the resistance on the convex side of the cups. The principal types of differential meters are the Price and the Ellis (Plates II and III).

The essentials for a good current meter are (*a*) simplicity in construction, with no delicate parts which easily get out of order; (*b*) a small area of resistance to the velocity of the water; (*c*) a simple and effective device for indicating the number of revolutions of the wheel; and (*d*) easy adaptability for use under all conditions.

Fig. 1 and Plates V and VI illustrate the difference in construction between meters of American and foreign make. The foreign meters are much more delicately constructed than the American, and their mechanism is more complicated, so that several men are needed to make a discharge measurement (Fig. 1, Plate VII) instead of one man, as with American meters (Plate VIII).

Although the small Price meter is the only one described herein, an effort has been made to adapt the general discussion to the use of current meters of any make.

Description of the Small Price Current Meter and Equipment.

The small Price current meter (Fig. 1) and equipment consists of five principal parts: (1) the head; (2) the tail; (3) the hanger and weights; (4) the recording device; and (5) the suspending device. In the following descriptions the numbers in parentheses refer to those on Fig. 1.

The Head.—The head consists of a ⊃-shaped yoke (1) carrying

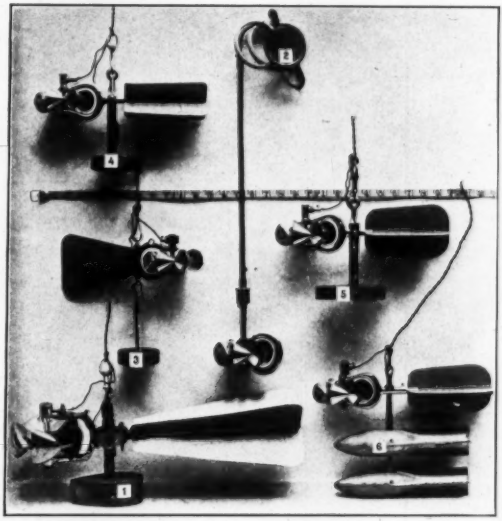


FIG. 1.—VARIOUS FORMS OF THE PRICE METER.

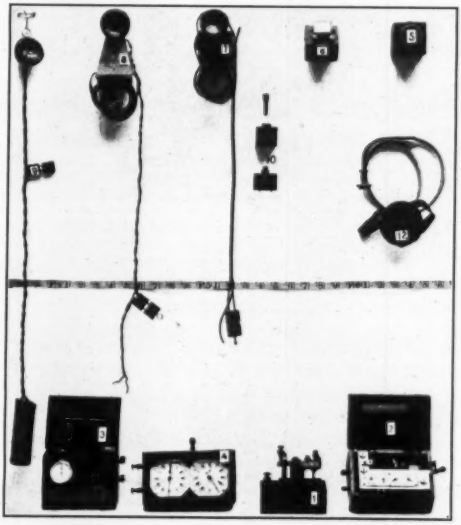


FIG. 2.—VARIOUS FORMS OF RECORDING DEVICES.

a wheel made of six conical cups (2), brazed to a horizontal frame (3). This wheel, referred to as the cups, turns on a vertical axis known as the cup shaft, which sets on a cone point bearing at the lower end and engages the recording mechanism at the upper end.

The cup shaft consists of two parts (4 and 5), screwed together from either side of the cup frame, thus fastening the cups rigidly to the cup shaft. At the lower part of the cup shaft there is a cone bearing which receives the cone point (6) on which the cups revolve. The cone point is screwed through a metal bushing (7) known as the cone plug, and is firmly held by a lock-nut (8). The cone plug fits into the lower arm of the yoke by a sliding connection, and is clamped in position by a set-screw. By means of a sleeve-nut (9) on the lower part of the shaft, the cups can be lifted from the cone point when the meter is not in use. This sleeve-nut has a left-handed thread, so that it will not tighten when the cups revolve. The upper part of the cup shaft is fitted with a worm gear which passes into a cylindrical chamber (10 or 10^a), known as the contact chamber, as it contains the mechanism for making the contact which indicates the revolutions of the cups. The construction and arrangement of both the contact chamber and the mechanism contained in it depend on whether an acoustic or an electric recording device is used.

When the electric recording device is used, the contact chamber (10) is closed by a screw cap (11) provided with a leather gasket for keeping out the water. It fits by a sliding connection into the upper end of the yoke, and is clamped into position by a set-screw. In the contact chamber there is fitted a cylindrical plug (12) which is held in position by a screw and carries a gear-wheel (13) which engages a worm gear on the upper end of the cup shaft, the gearing being arranged so that the wheel makes one revolution for every twenty revolutions of the cups. On the side of the wheel there are four platinum pins, equally spaced and set so that they will strike the contact spring (14) at each fifth revolution of the cups, thus closing the electric circuit, which is indicated by the recording device, as explained later. These contact parts are known as the contact wheel, the contact pins, and the contact spring. The contact spring is of platinum, and is carried by the contact plug (15) which is screwed into the contact chamber through a hard-rubber bushing (16), thus insulating the contact spring from the meter when it is not touching

one of the pins on the contact wheel. In the end of the contact plug there is a hole and a set-screw for connecting with a wire from the recording device.

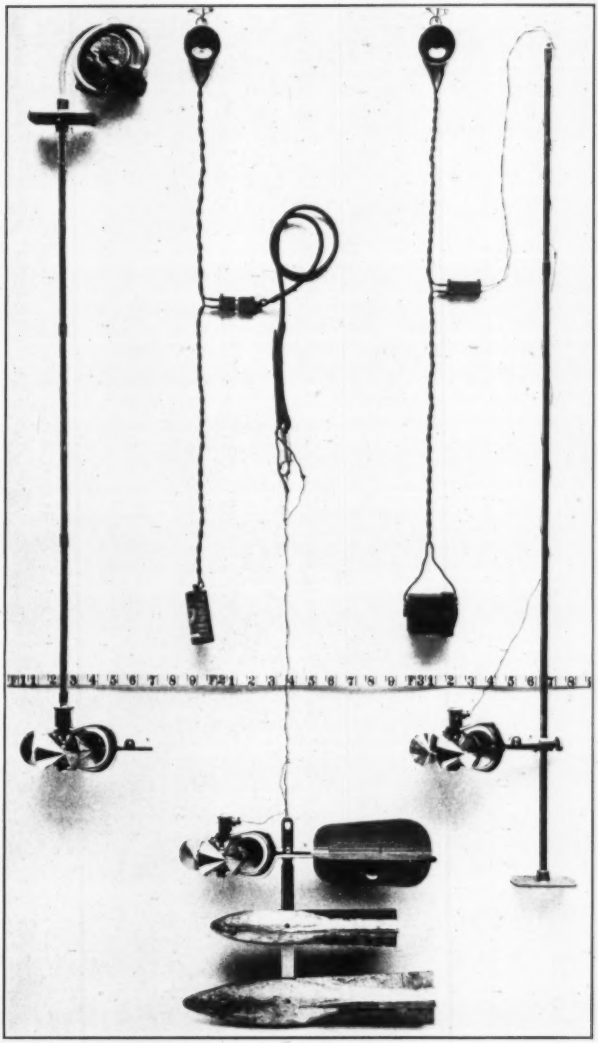
When the acoustic recording device is used, the contact chamber (10^a) is closed with a cap (11^a) and a metal drum (49), and, in place of the platinum contact spring (14) and the plug (16), there is a small hammer (50) which is arranged so that it is thrown against the drum by the pins on the side of the gear-wheel (13^a) at each fifth revolution of the cups.

The stem of the yoke contains a slot and a screw hole (22) for attaching the meter hanger (23), and a socket into which the stem of the tail of the meter (17) is fastened.

The Tail.—The tail is used when the meter is suspended by a cable. It provides for balancing the head, and also keeps the axis of the meter parallel to the direction of the current. It consists of a stem (17) which fits by a sliding connection into the socket in the stem of the yoke where it is clamped by a set-screw. On this stem there are two vanes (18 and 19) set at right angles. One of the vanes is rigidly attached to the stem; the other fits into it by grooves, so that it can readily be pulled out when the key (20) which holds it in place is turned. On one of the vanes there is a slot carrying a weight (21) which can be adjusted so as to balance the meter.

The Hanger and Weights.—When suspended with a cable, the meter is hung by a screw-bolt (22) on a steel stem (23) which passes through a slot in the stem of the yoke. The slot in the stem of the yoke is wide enough to allow the meter to swing freely in a vertical plane, and the bolt passes through the frame a little above the center of gravity of the meter, so that the latter will readily adjust itself to a horizontal position. In the upper end of the hanger there is a hole for attaching the suspending cable, and at intervals along the stem there are other holes by which the meter and lead weights may be hung. The weights (24) are of torpedo shape—this design offering the least resistance to the current—and are made in two sizes, weighing, respectively, 6½ and 15 lb. They are attached to the stem by a screw bolt. The manner of arrangement of the weights and meter on the stem depends on the conditions under which the measurements are to be made.

When the meter is used on a rod, the tail, hanger, and weight are



PRICE METERS AS NOW USED.

dispensed with, and the head is held by a sliding carrier (Plate IV), or it is attached to the rod (51) by a screw connection on the contact chamber when the acoustic recording device is used.

The set-screws for clamping the various sliding connections are all of the same size and of standard make. Beveled grooves are provided in each of these connections so that when the set-screws engage them the parts are drawn into place.

All parts of the meter are standard, and can readily be replaced in the field.

The Recording Device.—A recording or indicating device is necessary for determining the number of revolutions of the meter wheel, and the successful use of the meter depends primarily on this part of the apparatus.

Various devices, Fig. 2, Plate III, operated either on the mechanical, electric, or acoustic principle, have been used for this purpose. These include the telegraph ticker, electric buzzer, telephone receiver, drums, etc. Of these, however, the telephone attachment and the acoustic recorder have been found to be most satisfactory in general practice.

The telephone attachment, Fig. 1, consists of a telephone receiver (25) and small battery (26), connected in a circuit which terminates in a connecting plug (27) by which the apparatus can be readily connected or disconnected in circuit with the meter, depending on the method of suspension, as explained later.

The magnets of the telephone receiver are wound for 10-ohm resistance so as to make as loud a click as possible.

Either a dry-cell or a wet-cell battery may be used. The most satisfactory dry cell (26) is the No. N 118, "Ever Ready" cell, which is 1 in. in diameter and 3 in. long. Connection is made with this cell through two screw connecting posts (28), both at the same end of the cell.

The wet cell consists of an outer casing of hard rubber (29), about 1½ in. square, containing a carbon compartment (30) into which a zinc pole (31) having a rubber stopper (32) is inserted. The current is generated by a solution of bisulphate of mercury and water. Contact is made from the cell through a platinum plug (33) extending into the carbon at the bottom and through the screw (34) in the zinc pole which extends through the rubber stopper.

The cell is encased in a leather box (35), and connection is made

with it through two screw connecting posts (36), each of which terminates in a separate spring plate (37) against which the poles of the battery bear.

In use, the receiver is pinned to the left shoulder and the battery cell is placed in the side coat pocket. The connecting plug (27) will then hang a little below the shoulder and is easily accessible for attaching and detaching the meter.

In the acoustic recorder, the striking of the hammer (50) on the drum (49) in the contact chamber (10^a) indicates each fifth revolution of the meter, as already explained. The sound is transmitted through the rods (51) and a rubber tube to the ear of the observer. The rubber tube and ear-piece (Plate IV) are not necessary unless there is considerable noise at the station, as the click is loud enough to be heard without them.

Automatic recorders, Fig. 2, Plate III, have been used to some extent to determine the number of revolutions of the meter. For general work, however, these are not entirely satisfactory, as they are likely to get out of order, and are inconvenient to carry.

The time taken by the meter wheel to make a given number of revolutions should be determined with a stop-watch, as by this means the error in the observation can be reduced by observing to a fractional part of a second.

The Suspending Device.—The suspending device, which consists of a rod or of some form of cable, must make provision for lowering the meter and weight into the water and also for completing an electric circuit between the meter wheel and the recording device.

The rod in common use in connection with the electric recorder consists of a ½-in. tube (Plate IV) graduated to feet and tenths. For convenience in carrying, it is made in 1-ft. sections which screw together. On the bottom of the rod there is a flat foot which keeps it from sinking into the bottom of the stream, and at the top there is a plug for connecting one of the wires from the recording device. The head of the meter is attached to a sliding hanger, which can be moved up and down the rod or clamped in any position. The circuit between the meter wheel and the recording device is made by attaching one of the wires from the recording device to the plug in the top of the rod. The other wire follows down the rod, and is attached to the contact plug of the meter.

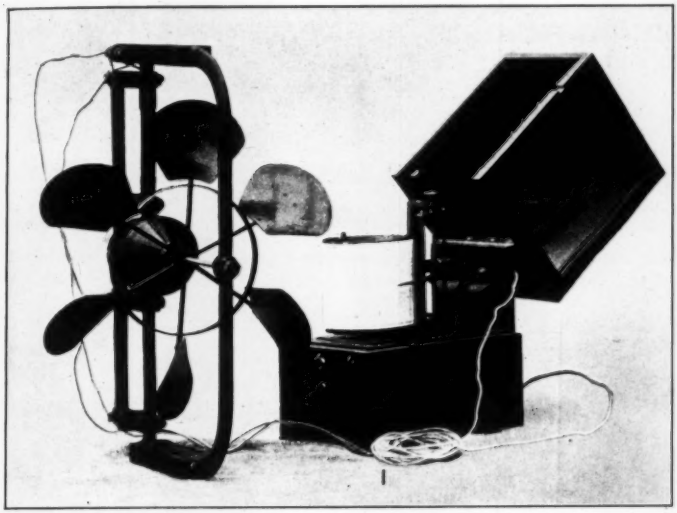


FIG. 1.—RICHARDS METER, WITH ELECTRICAL RECORDER.

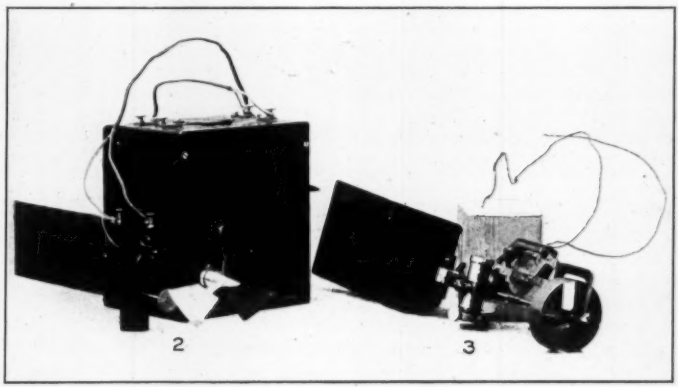


FIG. 2.—OTT AND WOLTMAN METERS, WITH ELECTRICAL RECORDERS.

In using the meter on the rod, the foot-piece of the rod is set on the bottom of the stream, and the head of the meter is adjusted so that it will come to the point at which it is desired to measure the velocity. By this means the meter can be held firmly in the required position.

The rods (Plate IV) for use with the acoustic recorder are of ½-in. tubing graduated to feet and tenths, and, for convenience in carrying, are made in 1-ft. sections which screw together. The bottom rod connects with the contact chamber with a screw, and is cut so that the distance from the center of the cups to the end of the rod is just 1 ft. On the upper end of the top rod there is a flat plate, in the center of which there is a hole through which the sound from the drum can be heard. The soundings are made with this end of the rod, and the plate keeps the end from sinking into the bed of the stream.

The best form of cable (Fig. 1) is a combination of No. 16, old code, double-insulated, show-window cord (38) and No. 12 or 14 galvanized wire (39) about which is wound a small insulated magnet wire (40).

The show-window cord is used for the upper part of the cable. It is large enough to be manipulated easily with bare hands, and, being made of two insulated wires, provides for making the direct and return circuit between the meter and the recording device. In its use, the two wires of which it is made must be separated at either end (41, 42, 43, 44) in order to make the attachment with the connecting plug (27) of the recording device at the upper end and with the galvanized wire (39) and magnet wire (40) which lead to the meter at the lower end. A ring or snap (45), into which the galvanized wire is looped, is fastened, either by a loop (46) or a knot (47), to the lower end.

In fastening the meter cable to the snap with a knot (47), a strip of adhesive tape is wound around the cable two or three times, about 1 ft. from the end, leaving about 6 in. of the tape at the beginning and end of the winding. The cable is then inserted through the snap or ring so that the snap bears on the adhesive tape, and a knot is tied in the cable about the snap (45) and drawn down as tight as possible. The ends of the adhesive tape (48) are then wound around the cable, one above and the other below the knot, to keep it from sliding. The outside covering of the cable can then be taken off

within 3 or 4 in. of the knot, and one wire fastened to the wires leading to the contact plug and to the hanger.

If the snap is held in a loop (46), a length of about 12 or 14 in. of the outside insulation is removed so that the wires can be doubled back and connected with those leading to the contact plug and hanger. The loop is first tied with string and then wound with adhesive tape, the tape being placed around the cable where the ring bears on it.

The galvanized and magnet wires (39 and 40) make up the lower end of the cable, and should be long enough to reach from the surface to the deepest point in the stream. Their use is advantageous because they offer small resistance to the moving water and thus prevent the meter from being carried down stream.

The galvanized wire (39) provides both for carrying the weight of the meter and conducting the electric current between the meter and the recording device. It is attached by ordinary loop connections to the snap in the lower end of the show-window cord and to the meter hanger (23). The direct circuit is made through it by the direct connection with the meter stem and by its connection at the upper end with one of the insulated wires (41) from the show-window cord.

The fine magnet wire (40), which provides for the return circuit between the meter and the recording device, is wound loosely around the galvanized wire, and may be fastened with tire tape. At the upper end it is connected with one of the insulated wires (42) from the show-window cord, and at the lower end with the contact plug (15) of the meter. In order to aid in preserving the insulation between the galvanized and magnet wires they should be shellacked; and, even with this precaution, it may be necessary to replace the magnet wire frequently.

If the velocities are not so great as to carry the meter down stream, the galvanized and fine magnet wire may be dispensed with. The snap (45) at the lower end of the show-window cord may be attached directly to the meter stem, the circuit being completed by attaching the insulated wires (41, 42) to the contact plug and to the screw on the meter hanger, respectively.

The meter may also be suspended by a single uninsulated galvanized wire, the return circuit being made through the ground and water. In using the single wire the connection is from the water to the meter through the contact point to the line, then to the battery and through the 'phone to the bridge or cable, then to the ground and back

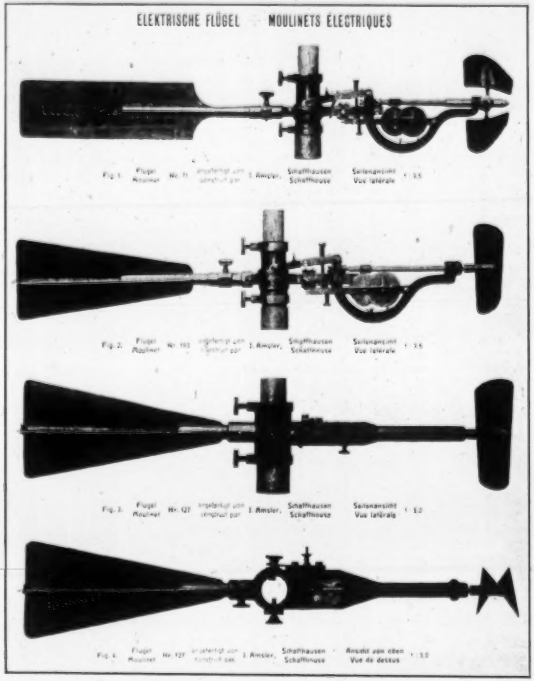


FIG. 1.—TYPES OF AMSLER METER USED IN THE SWISS HYDROGRAPHIC SERVICE.

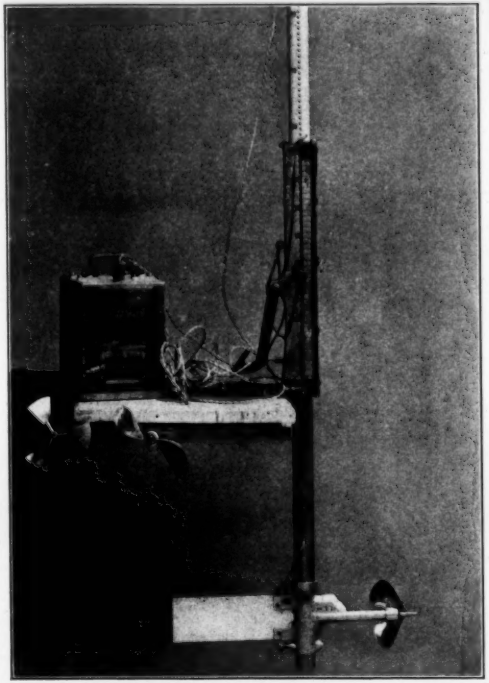


FIG. 2.—OTT METER USED IN THE FRENCH HYDROGRAPHIC SERVICE.

to the water. It does not make any difference on which side of the battery the 'phone is placed in the line (Fig. 2).

When using a single wire, on a cable or a bridge, a clean metallic contact should be made. A little paint, rust, or other coating will prevent good work.

In measuring high velocities and deep streams, where the suspending cables afford insufficient support, stay-lines or guy-lines are used to keep the meter in place.

Rating the Meter.

The relation between the revolutions of the meter wheel and the velocity of the water must be determined by rating each meter before it is used. Theoretically, the rating for all meters of the same make should be the same, but, as a result of slight variations in construction, and in the bearing of the wheel on the axis at different velocities, the ratings differ.

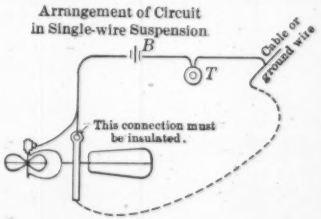


Fig. 2.

A meter is rated by conducting it through still water with uniform speed, and noting the time, the number of revolutions, and the distance. The revolutions per second and the velocity in feet per second are afterward computed from these data. Many runs are made, as shown in Table 1, the speeds varying from the least which will cause the wheel to revolve to several feet per second. The results of these runs, when plotted (Fig. 3) with revolutions per second and velocity in feet per second as co-ordinates, locate the points which define the meter rating curve from which the rating table is prepared, as described later.

The United States Geological Survey has three rating stations, one at Washington, D. C., one at Denver, Colo., and one at Los Angeles, Cal. (Fig. 2, Plate VII). These stations consist of a platform over the water carrying a track about 200 ft. long, on which a car for carrying the meter is run. The track is near the edge of the platform, and the meter is suspended in the water by a projecting arm. At the Denver and Washington stations the car is propelled by hand; at the Los Angeles station it is moved electrically.

The number of revolutions of the meter wheel are indicated on an electric recorder; the distance is obtained by an electrical mechanism, which is in circuit with the meter wheel, so that the exact distance for a given number of revolutions is obtained; and the time is taken by a stop-watch.

TABLE 1.—OBSERVATIONS FOR SMALL PRICE PENTA METER RATING. MADE DECEMBER 14TH, 1908, AT CHEVY CHASE LAKE, MARYLAND, BY HENSHAW AND WALTERS. METER HUNG WITH ONE 15-LB. AND ONE 6½-LB. TORPEDO WEIGHT.

No. of run.	Observations for Length of Run.			Time, in seconds	No. of revolutions.	Revolutions per second.	Velocity, in feet per second.	Remarks.
	Start.	End.	Distance.					
	Feet.	Feet.	Feet.					
1	— 6.8	32.0	25.2	79	10	0.127	0.319	
2	21.4	+ 2.9	24.3	65	10	0.154	0.374	
3	— 7.3	32.1	24.8	67	10	0.149	0.370	
4	25.0	— 0.4	24.6	62	10	0.161	0.397	
5	— 10.7	35.2	24.5	45	10	0.222	0.544	
6	28.2	— 2.8	25.4	50	10	0.200	0.508	
7	— 13.6	62.5	48.9	78	20	0.256	0.627	
8	54.2	— 5.1	49.1	77	20	0.260	0.638	
9	— 17.1	66.6	49.5	64	20	0.312	0.773	
10	61.4	— 12.9	48.5	68	20	0.294	0.713	
11	— 8.7	56.2	47.5	48	20	0.417	0.990	
12	52.1	— 5.0	47.1	50	20	0.400	0.942	
13	— 1.2	96.4	95.2	76	40	0.526	1.252	
14	83.7	+ 8.7	92.4	72	40	0.556	1.283	
15	+ 10.0	82.9	92.9	47	40	0.851	1.977	
16	90.0	+ 1.8	91.8	50	40	0.800	1.836	
17	+ 2.0	91.6	93.6	35	40	1.14	2.68	
18	90.7	— 0.3	90.4	29	40	1.38	3.12	
19	— 9.7	100.8	91.1	14.8	40	2.70	6.16	
20	95.9	— 6.3	89.6	17.4	40	2.30	5.15	
21	— 4.6	95.6	91.0	12.8	40	3.12	7.11	
22	97.4	— 6.8	90.6	11.6	40	3.45	7.81	
23	— 10.0	101.3	91.3	11.2	40	3.57	8.15	
24	83.7	+ 7.4	91.1	9.2	40	4.35	9.90	
25	0.0	92.7	92.7	9.0	40	4.44	10.30	
26	98.0	— 7.7	90.3	9.2	40	4.35	9.82	
27	+ 7.5	86.4	93.9	8.8	40	4.55	10.67	
28	94.5	— 1.6	92 9	7.8	40	5.13	11.91	
29	— 1.2	93.9	92.7	7.4	40	5.41	12.53	
30	100.6	— 9.1	91.5	7.8	40	5.13	11.73	
31	+ 2.4	91.6	94.0	7.0	40	5.71	13.43	
32	106.0	— 13.0	93.0	7.2	40	5.56	12.92	
33	+ 5.6	88.4	94.0	6.2	40	6.45	15.16	
34	101.1	— 7.9	93.2	6.6	40	6.06	14.12	

NOTE.—The runs are in pairs, the odd numbers being across the track and the even numbers in the return to the starting point.

Theoretically, the wheel of a differential-action meter, when carried through still water, should revolve as a wheel revolves in passing over the ground. That is, in going a given distance it should make practically the same number of revolutions, regardless of speed. The rating

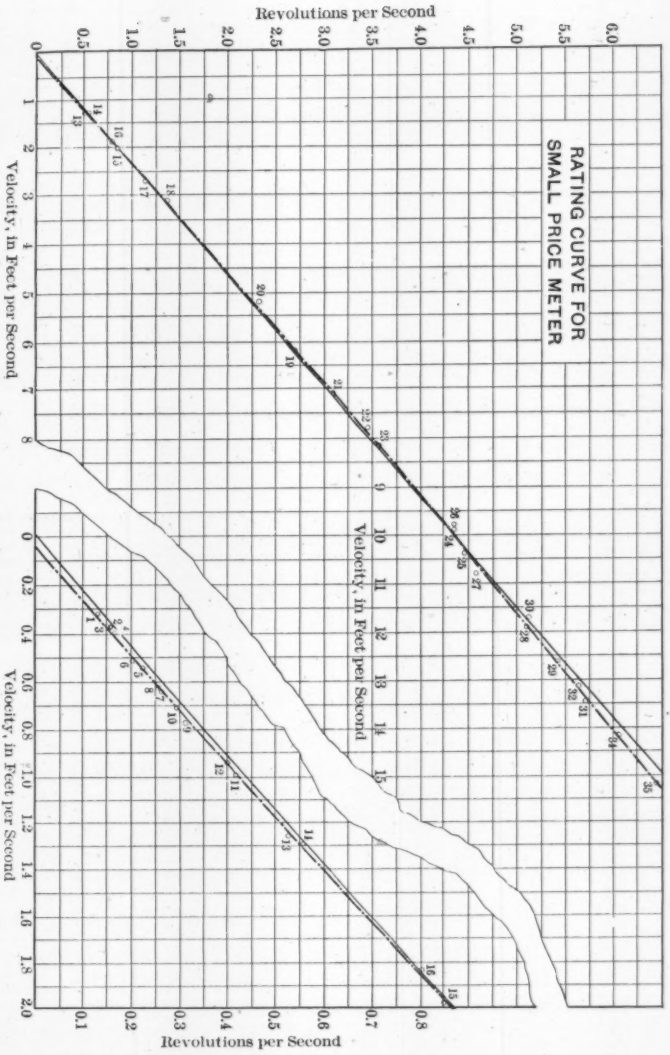

RATING CURVE FOR
SMALL PRICE METER
Revolutions per Second
Velocity, in Feet per Second
Velocity, in Feet per Second
Velocity, in Feet per Second
Revolutions per Second

FIG. 8.

of a great many small Price electric meters shows this number to be from 42 to 44 revolutions in going 100 ft.

The true number of revolutions of the wheel should equal the distance of the run divided by the effective circumference of the wheel multiplied by a coefficient which depends on the retarding effect due to the pressure on the convex surface of the cups and the blanketing effect of the cups. Assuming the effective circumference to be the circle passing through the points of the cups, which is 0.7854 ft., and the true number of revolutions to be 43½ per 100 ft. run; then the coefficient would be 0.345. Although complete data are not available to confirm this theory, the working of the meter shows that it holds very closely to it.

The foregoing shows that the theoretical meter-rating curve is a straight line passing through the origin. If the true number of revolutions made in going 100 ft. is 43½, the equation of this curve will be $X = 2.3\ Y$, where X = velocity, in feet per second, and Y = revolutions per second.

It is interesting to note that Dubuat found that the resistance of a circular plate, when drawn through still water, as compared with that of a sphere of the same diameter, was as 100 to 35.

A study of the rating curves of a large number of small Price meters shows that, as a rule, the curve is made up of two straight lines, the extension of the lower one joining the upper one in an angle between the velocities of 8 and 9 ft. At this point there is a slight increase in the friction on the bearings of the meter wheel and shaft. Notwithstanding this break in the curve, the observed curve parallels the theoretical curve very closely. The lower part of the curve starts at a velocity of less than 0.1 ft. per sec., which is required to start the wheel.

In plotting the rating curve, velocities in feet per second are generally shown as abscissas and revolutions per second as ordinates. The scale for velocities of less than 2 ft. per sec. should be 5 in. per ft. of velocity, and, for greater velocities, 1 in. per ft. of velocity. The scale for revolutions per second, for velocities of less than 2 ft., should be 10 in. per rev. per sec., and for velocities greater than 2 ft., it should be 2 in. per rev. per sec. This necessitates the construction of two curves: one for low velocities and one for high velocities. The low-velocity curve should be constructed first and two points taken from

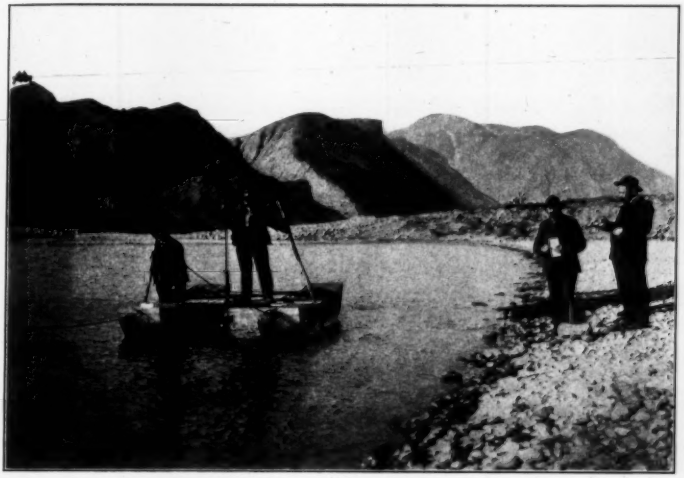


FIG. 1.—MEASUREMENTS AT BOAT GAUGING STATION, ON RIVER DURONÉE, NEAR ROUSSET, FRANCE.

FIG. 2.—U. S. GEOLOGICAL SURVEY CURRENT-METER RATING STATION, LOS ANGELES, CAL.

this curve should be re-plotted at the smaller scale, in order to determine the position of the lower portion of the curve. The theoretical, or a mean, rating curve should also be drawn on the sheet as a guide for drawing the curve.

In using the meter, observation is made of the number of seconds the wheel requires to make a given number of revolutions. Therefore, a rating table is prepared for each meter, giving the velocities per second corresponding to the number of revolutions. The most convenient table is prepared for 5, 10, 20, 30, 40, 50, 60, 70, 80, 90, 100, 150, and 200 revolutions, with the times of the runs ranging from 30 to 60 sec., giving velocities from 0.234 to 15.69 ft. per sec.

The rating table (Table 2) is prepared by tabulating the number of revolutions per second from 30 to 60 sec. for each length of run, as shown in the column headed "Revolutions per second." These values enable taking from the curve, Fig. 3, the velocity per second for each number of seconds run.

As the curves for various meters are practically parallel, differences for consecutive velocities on each are practically the same. Therefore, it is only necessary to take occasional velocities from the curve, and intermediate values can be filled in by adding the successive differences from a mean table, as given in the column headed "Difference." These differences also aid in using the table to interpolate for fractions of a second.

In preparing the mean rating curve and table from which the values in the column headed "Difference" have been obtained, the mean of the ratings of nine meters was taken, the rating table, illustrated by Tables 1 and 2 and Fig. 3, is for a current meter having a rating which is identical with this mean curve and table. Such coincidences often appear, either for the whole curve or for a portion of it.

Table 3 gives the percentage of deviation from the mean curve of the different curves used in determining it. An examination of this table shows that the error introduced, due to differences in the ratings of the meters, is very small, and that for meters in good condition the standard rating table can be used without serious error for all velocities greater than 0.5 ft. per sec. The comparison of numerous other rating curves with the standard shows practically the same percentage of variation.

TABLE 2—*Continued.*

41	0.122	0.322	0.007	0.244	0.599	0.013	0.488	1.15	0.03	0.732	1.71	0.04	0.976	2.26	0.05	1.22	2.82	0.07	2.44	5.60	0.13	41
42	0.119	0.315	0.006	0.238	0.586	0.013	0.476	1.13	0.03	0.714	1.67	0.04	0.952	2.21	0.05	1.19	2.75	0.06	2.38	5.46	0.13	42
43	0.116	0.309	0.006	0.233	0.573	0.012	0.465	1.10	0.02	0.698	1.63	0.04	0.930	2.16	0.05	1.16	2.69	0.06	2.33	5.34	0.12	43
44	0.114	0.303	0.006	0.227	0.561	0.011	0.455	1.08	0.02	0.682	1.60	0.03	0.909	2.11	0.05	1.14	2.63	0.06	2.27	5.22	0.11	44
45	0.111	0.297	0.005	0.222	0.550	0.011	0.444	1.06	0.02	0.667	1.56	0.03	0.889	2.07	0.04	1.11	2.57	0.05	2.22	5.10	0.11	45
46	0.109	0.291	0.005	0.217	0.539	0.011	0.435	1.03	0.02	0.652	1.53	0.03	0.870	2.02	0.04	1.09	2.52	0.05	2.17	4.99	0.11	46
47	0.106	0.286	0.005	0.213	0.528	0.010	0.426	1.01	0.020	0.638	1.50	0.03	0.851	1.98	0.04	1.06	2.47	0.05	2.13	4.89	0.10	47
48	0.104	0.281	0.005	0.208	0.518	0.010	0.417	0.992	0.019	0.625	1.47	0.03	0.833	1.94	0.04	1.04	2.41	0.05	2.08	4.79	0.10	48
49	0.102	0.276	0.005	0.204	0.508	0.009	0.408	0.973	0.019	0.612	1.44	0.03	0.816	1.90	0.04	1.02	2.37	0.05	2.04	4.69	0.09	49
50	0.100	0.272	0.004	0.200	0.499	0.009	0.400	0.954	0.018	0.600	1.41	0.03	0.800	1.86	0.04	1.00	2.32	0.04	2.00	4.60	0.09	50
51	0.098	0.267	0.004	0.196	0.490	0.009	0.392	0.937	0.017	0.588	1.38	0.03	0.784	1.83	0.03	0.980	2.28	0.04	1.96	4.51	0.09	51
52	0.096	0.263	0.004	0.192	0.482	0.008	0.385	0.919	0.017	0.577	1.36	0.02	0.769	1.79	0.03	0.962	2.23	0.04	1.92	4.42	0.08	52
53	0.094	0.259	0.004	0.189	0.473	0.008	0.377	0.903	0.016	0.566	1.33	0.02	0.755	1.76	0.03	0.943	2.19	0.04	1.89	4.34	0.08	53
54	0.093	0.255	0.004	0.185	0.465	0.008	0.370	0.887	0.015	0.556	1.31	0.02	0.741	1.73	0.03	0.926	2.15	0.04	1.85	4.26	0.08	54
55	0.091	0.251	0.004	0.182	0.458	0.007	0.364	0.872	0.015	0.545	1.29	0.02	0.727	1.70	0.03	0.909	2.11	0.04	1.82	4.18	0.07	55
56	0.089	0.247	0.004	0.179	0.450	0.007	0.357	0.857	0.014	0.536	1.26	0.02	0.714	1.67	0.03	0.893	2.08	0.04	1.79	4.11	0.07	56
57	0.088	0.244	0.003	0.175	0.443	0.007	0.351	0.843	0.014	0.526	1.24	0.02	0.702	1.64	0.03	0.877	2.04	0.03	1.75	4.04	0.07	57
58	0.086	0.240	0.003	0.172	0.436	0.007	0.345	0.829	0.013	0.517	1.22	0.02	0.690	1.61	0.03	0.862	2.01	0.03	1.72	3.97	0.07	58
59	0.085	0.237	0.003	0.169	0.430	0.006	0.339	0.816	0.013	0.508	1.20	0.02	0.678	1.59	0.03	0.847	1.97	0.03	1.69	3.90	0.06	59
60	0.083	0.234		0.167	0.423		0.333	0.803		0.500	1.18		0.667	1.56		0.833	1.94		1.67	3.84		60

NOTE.—Tabulated values as given above are taken from computations carried to six significant figures. This accounts for the differences in the velocity column not always agreeing in the last place with those given in the difference column.

TABLE 2.—MEAN RATING TABLE FOR SMALL PRICE METER.

Based on Ratings of Nine Meters, Made During 1908, at Chevy Chase Lake, Maryland.

Equation Curve { Between 0.0 and 3.9 Rev. per Sec. $x = 2.27600y + 0.04400$
" 3.9 " 7.0 " " $x = 2.44333y - 0.60332$

Time, in Seconds.	5 Revolutions.			10 Revolutions.			20 Revolutions.			30 Revolutions.			40 Revolutions.			50 Revolutions.			100 Revolutions.			Time, in Seconds.
	Revolutions per Second.	Velocity per Second.	Difference.	Revolutions per Second.	Velocity per Second.	Difference.	Revolutions per Second.	Velocity per Second.	Difference.	Revolutions per Second.	Velocity per Second.	Difference.	Revolutions per Second.	Velocity per Second.	Difference.	Revolutions per Second.	Velocity per Second.	Difference.	Revolutions per Second.	Velocity per Second.	Difference.	
30	0.167	0.433		0.333	0.808		0.667	1.56		1.00	2.32		1.33	3.08		1.67	3.84		3.33	7.63		**30**
31	0.161	0.411	0.022	0.323	0.778	0.030	0.645	1.51	0.05	0.968	2.25	0.07	1.29	2.98	0.10	1.61	3.71	0.12	3.23	7.39	0.24	**31**
32	0.156	0.400	0.011	0.312	0.755	0.023	0.625	1.47	0.04	0.938	2.18	0.07	1.25	2.89	0.09	1.56	3.60	0.11	3.12	7.16	0.23	**32**
33	0.152	0.389	0.011	0.303	0.734	0.021	0.606	1.42	0.04	0.909	2.11	0.06	1.21	2.80	0.09	1.52	3.49	0.11	3.03	6.94	0.22	**33**
34	0.147	0.379	0.010	0.294	0.713	0.020	0.588	1.38	0.04	0.882	2.05	0.06	1.18	2.72	0.08	1.47	3.39	0.10	2.94	6.74	0.20	**34**
35	0.143	0.369	0.010	0.286	0.694	0.019	0.571	1.34	0.04	0.857	1.99	0.05	1.14	2.65	0.07	1.43	3.30	0.09	2.86	6.55	0.18	**35**
36	0.139	0.360	0.009	0.278	0.676	0.018	0.556	1.31	0.03	0.833	1.94	0.05	1.11	2.57	0.07	1.39	3.21	0.09	2.78	6.37	0.17	**36**
37	0.135	0.352	0.008	0.270	0.659	0.017	0.541	1.27	0.03	0.811	1.89	0.05	1.08	2.50	0.07	1.35	3.12	0.09	2.70	6.20	0.17	**37**
38	0.132	0.343	0.008	0.263	0.643	0.016	0.526	1.24	0.03	0.789	1.84	0.05	1.05	2.44	0.06	1.32	3.04	0.08	2.63	6.03	0.16	**38**
39	0.128	0.335	0.008	0.256	0.628	0.015	0.513	1.21	0.03	0.769	1.79	0.05	1.03	2.38	0.06	1.28	2.96	0.08	2.56	5.88	0.15	**39**
40	0.125	0.328	0.007	0.250	0.613	0.015	0.500	1.18	0.03	0.750	1.75	0.04	1.00	2.32	0.06	1.25	2.89	0.07	2.50	5.73	0.14	**40**

TABLE 2—*Continued.*

46	1.30	3.01	0.06	1.52	3.51	0.07	1.74	4.00	0.08	1.96	4.50	0.09	2.17	4.99	0.11	3.26	7.47	0.16	4.35	10.02	0.23	**46**
47	1.28	2.95	0.06	1.49	3.43	0.07	1.70	3.92	0.08	1.91	4.40	0.09	2.13	4.89	0.10	3.19	7.31	0.15	4.26	9.79	0.22	**47**
48	1.25	2.89	0.06	1.46	3.36	0.07	1.67	3.84	0.08	1.88	4.31	0.09	2.08	4.79	0.10	3.12	7.16	0.15	4.17	9.58	0.21	**48**
49	1.22	2.83	0.06	1.43	3.30	0.07	1.63	3.76	0.07	1.84	4.22	0.09	2.04	4.69	0.10	3.06	7.01	0.14	4.08	9.37	0.20	**49**
50	1.20	2.78	0.05	1.40	3.23	0.06	1.60	3.69	0.07	1.80	4.14	0.08	2.00	4.60	0.09	3.00	6.87	0.13	4.00	9.17	0.19	**50**
51	1.18	2.72	0.05	1.37	3.17	0.06	1.57	3.61	0.07	1.76	4.06	0.08	1.96	4.51	0.09	2.94	6.74	0.13	3.92	8.98	0.18	**51**
52	1.15	2.67	0.05	1.35	3.11	0.06	1.54	3.55	0.07	1.73	3.98	0.07	1.92	4.42	0.09	2.88	6.61	0.12	3.85	8.80	0.17	**52**
53	1.13	2.62	0.05	1.32	3.05	0.06	1.51	3.48	0.06	1.70	3.91	0.07	1.89	4.34	0.08	2.83	6.49	0.12	3.77	8.63	0.16	**53**
54	1.11	2.57	0.05	1.30	2.99	0.05	1.48	3.42	0.06	1.67	3.84	0.07	1.85	4.26	0.08	2.78	6.37	0.11	3.70	8.47	0.15	**54**
55	1.09	2.53	0.04	1.27	2.94	0.05	1.45	3.35	0.06	1.64	3.77	0.07	1.82	4.18	0.07	2.73	6.25	0.11	3.64	8.32	0.15	**55**
56	1.07	2.48	0.04	1.25	2.89	0.05	1.43	3.30	0.06	1.61	3.70	0.06	1.79	4.11	0.07	2.68	6.14	0.11	3.57	8.17	0.14	**56**
57	1.05	2.44	0.04	1.23	2.84	0.05	1.40	3.24	0.06	1.58	3.64	0.06	1.75	4.04	0.07	2.63	6.03	0.10	3.51	8.03	0.14	**57**
58	1.03	2.40	0.04	1.21	2.79	0.05	1.38	3.18	0.05	1.55	3.58	0.06	1.72	3.97	0.07	2.59	5.93	0.10	3.45	7.89	0.13	**58**
59	1.02	2.36	0.04	1.19	2.74	0.05	1.36	3.13	0.05	1.53	3.52	0.06	1.69	3.90	0.06	2.54	5.83	0.10	3.39	7.76	0.13	**59**
60	1.00	2.32		1.17	2.70		1.33	3.08		1.50	3.46		1.67	3.84		2.50	5.73		3.33	7.63		**60**

NOTE.—Tabulated values as given above are taken from computations carried to six significant figures. This accounts for the differences in the velocity column not always agreeing in the last place with those given in the difference column.

TABLE 2—*Continued.*

Time, in Seconds.	60 Revolutions.			70 Revolutions.			80 Revolutions.			90 Revolutions.			100 Revolutions.			150 Revolutions.			200 Revolutions.			Time, in Seconds.
	Revolutions per Second.	Velocity per Second.	Difference.	Revolutions per Second.	Velocity per Second.	Difference.	Revolutions per Second.	Velocity per Second.	Difference.	Revolutions per Second.	Velocity per Second.	Difference.	Revolutions per Second.	Velocity per Second.	Difference.	Revolutions per Second.	Velocity per Second.	Difference.	Revolutions per Second.	Velocity per Second.	Difference.	
30	2.00	4.60		2.33	5.35		2.67	6.11		3.00	6.87		3.33	7.63		5.00	11.61		6.67	15.69		30
31	1.94	4.45	0.15	2.26	5.18	0.17	2.58	5.92	0.20	2.90	6.65	0.22	3.23	7.39	0.24	4.84	11.22	0.39	6.45	15.16	0.53	31
32	1.88	4.31	0.14	2.19	5.02	0.16	2.50	5.73	0.18	2.81	6.45	0.21	3.12	7.16	0.23	4.69	10.85	0.37	6.25	14.67	0.49	32
33	1.82	4.18	0.13	2.12	4.87	0.15	2.43	5.56	0.17	2.73	6.25	0.19	3.03	6.94	0.22	4.55	10.50	0.35	6.06	14.20	0.46	33
34	1.76	4.06	0.12	2.06	4.73	0.14	2.35	5.40	0.16	2.65	6.07	0.18	2.94	6.74	0.20	4.41	10.16	0.33	5.88	13.77	0.41	34
35	1.71	3.95	0.11	2.00	4.60	0.13	2.29	5.25	0.15	2.57	5.90	0.17	2.86	6.55	0.19	4.29	9.87	0.31	5.71	13.06	0.39	35
36	1.67	3.84	0.10	1.94	4.47	0.12	2.22	5.10	0.14	2.50	5.73	0.16	2.78	6.37	0.18	4.17	9.58	0.29	5.56	12.97	0.37	36
37	1.62	3.73	0.10	1.89	4.35	0.12	2.16	4.97	0.14	2.43	5.58	0.15	2.70	6.20	0.17	4.05	9.30	0.28	5.41	12.60	0.37	37
38	1.58	3.64	0.10	1.84	4.24	0.11	2.11	4.84	0.13	2.37	5.43	0.15	2.63	6.03	0.16	3.95	9.04	0.26	5.26	12.26	0.35	38
39	1.54	3.55	0.09	1.79	4.13	0.11	2.05	4.71	0.12	2.31	5.30	0.14	2.56	5.88	0.15	3.85	8.80	0.24	5.13	11.93	0.33	39
40	1.50	3.46	0.09	1.75	4.03	0.10	2.00	4.60	0.12	2.25	5.16	0.13	2.50	5.73	0.15	3.75	8.58	0.22	5.00	11.61	0.31	40
41	1.46	3.37	0.08	1.71	3.93	0.10	1.95	4.48	0.11	2.20	5.04	0.12	2.44	5.60	0.14	3.66	8.37	0.21	4.88	11.32	0.30	41
42	1.43	3.30	0.08	1.67	3.84	0.09	1.90	4.38	0.11	2.14	4.92	0.12	2.38	5.46	0.13	3.57	8.17	0.20	4.76	11.08	0.28	42
43	1.40	3.22	0.08	1.63	3.75	0.09	1.86	4.28	0.10	2.09	4.81	0.11	2.33	5.34	0.13	3.49	7.98	0.19	4.65	10.76	0.27	43
44	1.36	3.15	0.07	1.59	3.66	0.08	1.82	4.18	0.10	2.05	4.70	0.11	2.27	5.22	0.12	3.41	7.80	0.18	4.55	10.50	0.26	44
45	1.33	3.08	0.07	1.56	3.58	0.08	1.78	4.09	0.09	2.00	4.60	0.10	2.22	5.10	0.11	3.33	7.69	0.16	4.44	10.26	0.24	45

TABLE 3.—Deviation of Velocity in Individual Current Meter Rating Tables from that of the Mean Table, Expressed as a Percentage of the Velocity of the Mean Table.

Revolutions per Second.	Velocity, Mean Table.	Meter Number.								
		738	739	788	789	790	792	793	794	795
	Feet per second.	Per cent.	Per cent.	Per cent.	Per cent.	Per cent.	Per cent.	Per cent.	Per cent.	Per cent.
0.1	0.269	−2.6	−1.5	−2.2	−0.74	−3.3	+7.4	+3.3	−3.0	+0.74
0.2	0.494	+0.40	−1.4	−0.20	−0.81	−1.8	+4.4	+1.8	−1.6	0.0
0.3	0.721	+1.1	−1.8	0.0	−1.5	−1.2	+3.0	+1.4	−1.1	0.0
0.4	0.945	+1.8	−1.6	+0.1	−1.4	−0.95	+2.3	+0.95	−0.95	−0.32
0.5	1.17	0.0	−1.7	0.0	−0.85	−0.85	+1.7	+0.85	−0.85	0.0
0.6	1.39	0.0	−1.4	+0.72	−0.72	0.0	+2.2	+1.4	−0.72	0.0
0.7	1.62	0.0	−1.2	0.0	−0.62	−0.62	+1.2	0.0	−0.62	0.0
0.8	1.84	0.0	−0.54	+1.1	−0.54	+0.54	+1.6	+0.54	0.0	−0.54
0.9	2.07	0.0	−0.97	+0.48	−0.48	0.0	+1.9	+0.97	0.0	0.0
1.0	2.30	−0.43	−0.87	+0.87	−0.43	0.0	+1.1	+1.7	+0.43	−0.87
1.5	3.44	−1.2	−0.58	+0.29	−0.87	−0.29	+0.87	+1.2	+0.29	−0.58
2.0	4.59	−0.87	−0.87	+0.22	−1.3	−0.44	+1.1	+1.1	+0.44	0.0
2.5	5.73	−0.35	−0.70	+0.35	−1.2	−0.35	+1.0	+1.2	+0.52	+1.7
3.0	6.87	−0.15	−1.0	0.0	−1.3	−0.15	+1.0	+1.2	+0.58	+0.44
3.5	8.01	+0.12	−1.0	+0.12	−1.4	+0.75	+0.62	+0.62	+0.37	+0.25
4.0	9.17	0.0	.....	+0.54	−0.11	−0.33	.....	.....	.....	.....
4.5	10.38	−0.48	.....	+0.29	+0.48	−0.10	.....	.....	.....	.....
5.0	11.58	−0.86	.....	+0.34	+0.69	−0.09	.....	.....	.....	.....
5.5	12.82	.....	.....	−0.23	+0.62	−0.31	.....	.....	.....	.....
6.0	14.06	.....	.....	−0.50	+0.71	−0.28	.....	.....	.....	.....
6.5	15.26	.....	.....	−0.52	+0.59	.....	.....	.....	.....	.....

Care of the Current Meter.

Although the current meter is substantially made, and will stand considerable hard usage, it needs special attention and care to insure its proper working.

In taking the meter apart, remove the tail vanes and the hanger stem; then loosen the set-screw to the contact chamber, and pull the chamber out by a slight twisting motion. Care must be taken to let the cups be free to turn, so that the worm gear on the upper end of the shaft can disengage itself from the teeth of the contact wheel. In handling the contact chamber, it is well to take off the cap, so that the gear-wheel can be seen during the operation. The cone point can then be taken out and the cups released by loosening the upper part of the shaft with a spanner wrench. This wrench is arranged so that all parts of the meter can be loosened with it.

In putting the meter together, first attach the cups to the cup shaft. In doing this, the upper part of the shaft should be inserted

through the upper hole of the yoke before it is screwed to the lower part. Care must be taken to place the cups so that they will move counter-clockwise. After the cups have been fastened to the shaft, insert the cone point and clamp it in place, and then insert the contact chamber. In replacing the contact chamber, the cups should be left free to move on the cone point, and care should be taken not to allow the cogs on the worm gear to catch on the teeth of the contact wheel. Before inserting the cone plug, the cone point should be adjusted and firmly secured with a lock-nut. The adjustment should allow a slight vertical motion of the cups.

There are two cases in which the meter can be carried: one is a box, 19 by 7 by 5 in., and the other is a box, 8½ by 6½ by 5 in. The larger box takes the meter with the tail attached, and allows sufficient room for carrying a small weight, a cable for suspending the meter, and miscellaneous tools. The small case is arranged with a shoulder-strap, and is just large enough to carry the meter and the tail when taken apart. The weights, the cable, and other equipment have to be carried in a separate case.

The following articles should accompany each current-meter equipment: A small screw-driver, a pair of parallel pliers with a wire cutter, a spanner wrench, an oil-can, a roll of adhesive tire tape, 25 ft. of metallic tape, a 50-ft. steel tape, an extra cone point, and extra set-screws.

In caring for and using a current meter, the points noted in the following paragraphs should be carefully considered:

1.—Be sure that the set-screws are all tightened before putting the meter in the water; otherwise one of the parts may be lost.

2.—Loosen the sleeve-nut and see that the meter runs freely before beginning a measurement; and spin the meter cups occasionally during a measurement to see that they are running freely.

3.—See that the weights play freely on the stem, so as to take the direction of the current and thus avoid a drag on the line.

4.—If any apparent inconsistency in the results of an observation throws doubt on its accuracy, investigate the cause at once. Grass may be wound around the cup shaft; the cups may be tilted by tension on the contact wire; the channel may be obstructed immediately above the meter; the meter may be in a hole; or the cups may be bent so as to come in contact with the yoke.

5.—After a measurement, clean and oil the bearings (in order to prevent rust) and inspect the cone point.

6.—In packing the meter, turn the sleeve-nut to lift the cups from the cone point.

7.—Always see that the lock-nut on the cone point is firmly screwed against the cone plug.

8.—If the cone point is dulled, it can be sharpened with an oil-stone.

9.—In measuring low velocities, be sure that the meter is in a horizontal position. If it has a tendency to tip, it can be held in place by using a plug in the slot for the stem.

10.—Avoid taking measurements in velocities of less than ½ ft. per sec., as the accuracy of the meter diminishes as zero velocity is approached. Compare velocities of 0.5 ft. per sec. and less, as shown in Table 3.

11.—For velocities of less than 1 ft. per sec., the bearing point should be sharp and smooth, as at the time of rating. As the velocity increases, the condition of the point is less important, for then the friction becomes a small factor.

12.—In taking measurements at high velocities, sufficient weight, or a stay-line, should be used to hold the hanger so that the meter will remain horizontal.

13.—In very shallow streams the meter should be suspended from the lower hole on the stem, and the weight should be placed above.

14.—If the cups of a small Price meter are bent, they may be easily put in shape by using a wood or metal bar with a round, smooth end.

15.—The telephone receiver is very sensitive to electric currents, and can be used to locate any break in the circuit. First try the telephone and battery together (Fig. 4) in a circuit having a make-and-break point, as at *a*. This may be done by using a knife blade or a screw-driver, making connection where the wires enter the plug. If there is no click in the 'phone, then the battery or the 'phone does not make a circuit. If there is a click, insert the meter in the line and test for a contact in the meter head (Fig. 5) by revolving the meter wheel. If the meter is all right, put the meter cord in the circuit and test both sides by making double connection and touching alternate sides of the line, *a* (Fig. 6).

16.—When the meter is not in use, disconnect the meter line from the battery, so that it will not become exhausted.

17.—When a measurement has been completed, the solution may be left in the wet cell if the zinc pole and stopper are replaced by a rubber cork.

18.—Never let the bisulphate dry in the cell, as it forms a hard cake and polarizes the battery.

19.—Do not let any bisulphate of mercury remain loose in the meter box; if it gets into the meter bearings it will corrode them.

20.—The zinc pole in the bisulphate cell sometimes gets pushed down so that it touches the bottom of the cell, in which case the cell is short-circuited and the recorder will not indicate. To test this, lift the plug a little way out of the battery and see if there is a flow of current.

METHODS OF TESTING CIRCUIT

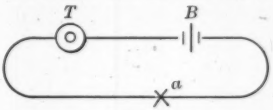


FIG. 4.

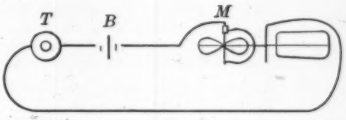


FIG. 5.

21.—Keep the points clean where the battery makes contact with the metal plates.

22.—The amount of current necessary to work with a telephone receiver is very small, and a battery may be serviceable even though nearly exhausted.

23.—If care is taken, it is very improbable that the telephone receiver will get out of order.

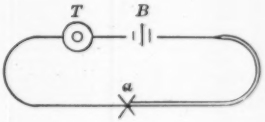


FIG. 6.

24.—Do not strike the telephone receiver, as a heavy jar will to a greater or less extent demagnetize the pole pieces, and to that extent will injure the receiver.

25.—Care must be taken not to short-circuit the dry battery when the meter is not in use, as in that way the cell becomes exhausted in a short time, the energy being used in heating the cell. To avoid this, the poles are wound with adhesive tape.

26.—If a dry cell which has been long in stock fails to work well, punch two nail holes in the wax on top of the cell and put it in water

over night, when it will absorb enough moisture to renew it. The holes should then be coated over by heating the wax with a match and pressing it into place, or by pouring in melted paraffin. A cell which has been exhausted by use is not benefited much by this treatment. The life of a cell depends largely on the amount of leakage in the line during use.

Discharge Measurements with the Current Meter.*

The measurement of stream discharge, for which the current meter was devised, consists in the determination of the quantity of water flowing past a given section of a stream in a given time. This quantity is obtained as the product of two factors: (*a*) The area of the cross-section of the stream, which is determined by sounding; and (*b*) the velocity of the water, which is measured with the current meter. The usual unit for expressing the discharge of a stream is the second-foot—an abbreviation for cubic foot per second—equivalent to the quantity of water flowing in a stream 1 ft. wide and 1 ft. deep, at a velocity of 1 ft. per sec.

Sections at which discharge measurements are regularly made are known as gauging stations, and may be divided into four classes, depending on whether the observations of velocity and depth are made from (*a*) a bridge, (*b*) a cable, (*c*) a boat, or (*d*) by wading (Plates VIII and IX). The method of making measurements is the same for all stations.

The ideal measuring section should be perpendicular to the thread of the velocity of the water and at a point on the stream where the conditions of bank and bed, both above and below the section, are permanent. This section, Fig. 7, is divided into partial areas bounded by perpendiculars terminating at points in the surface. These are known as measuring points, as the observations of the depth of the stream and the mean velocity of the water in the vertical are made at such points. The distance between the measuring points will depend on the size and character of the stream. The points should be spaced so as to show any irregularities either in the cross-section or in the velocity. If the section is to be used for future measurements, the measuring points should be indicated by permanent marks placed

* A detailed description of the collection and use of stream-flow data and the preparation of estimates of discharge may be found in "River Discharge," by Hoyt and Grover, published by Wiley & Sons, New York.

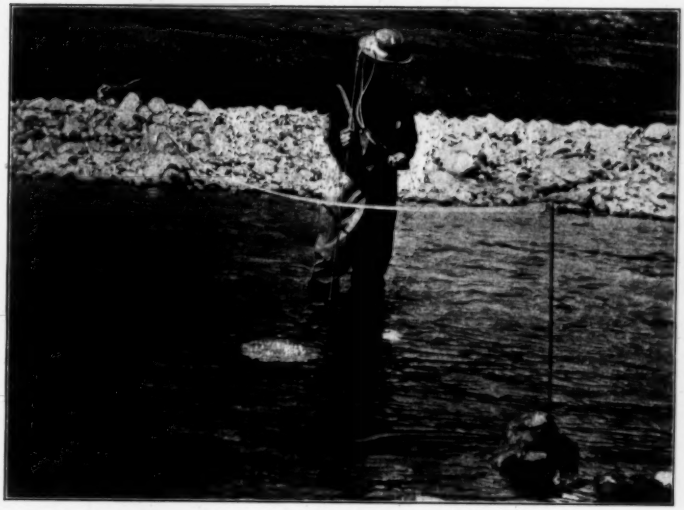


FIG. 1.—MEASUREMENT BY WADING, GRAND CENTRAL RIVER, ALASKA.

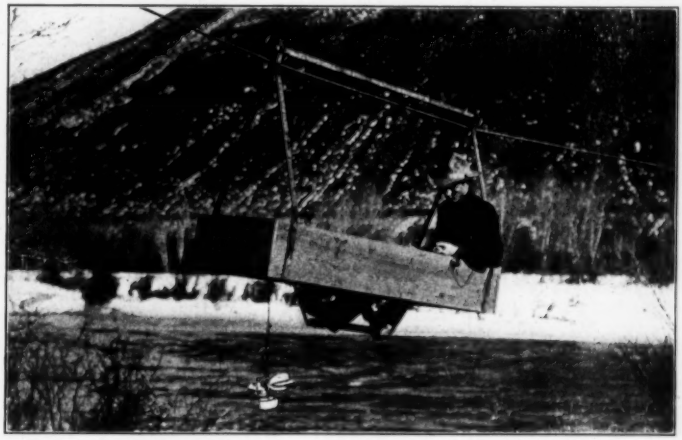


FIG. 2.—MEASUREMENT AT CABLE GAUGING STATION, YAKIMA RIVER, NEAR UNION GAP, WASH.

at intervals of 5 or 10 ft. on the bridge rail or floor of a bridge station, on the cable of a cable station, or on the tape or tagged line usually stretched across a stream when the measurements are made from a boat or by wading.

The procedure in the measurements varies somewhat with the sounding appliance. If the soundings are made with meter and cord, the depth and velocity are usually observed at each successive measuring point across the stream. If a rod or sounding line is used without the meter, soundings are made at all the measuring points before taking the velocities.

TYPICAL CROSS-SECTION OF STREAM
TO ILLUSTRATE DISCHARGE MEASUREMENT

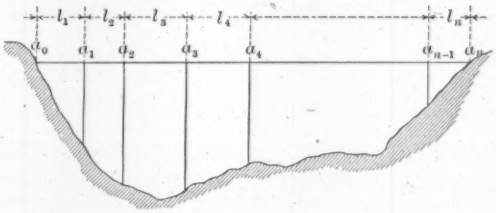


FIG. 7.

In sounding with the rod, care is taken that it does not sink into the bed of the stream, and that the reading is not too high on account of the water running up on it. The soundings from bridges or cables are usually made with the weight and line, and in such cases care is taken to eliminate the error due to the weight being carried down the stream or to the bowing of the line. Soundings with the line are most readily taken as follows: The weight and line are lowered until the weight rests on the bed of the river directly under the measuring point, and, with the line taut, a point is marked on it opposite a fixed point on the bridge or car; the weight is then raised until it just touches the surface of the water and the length of the sounding line that passes the fixed point is measured. The depth is measured by placing the end of a linen or metallic tape opposite the fixed starting point on the sounding line, grasping both the line and the tape in the hands, and drawing up the line and tape without permitting them to slip on each other until the weight reaches the surface of the water. The length of line thus drawn up, representing the depth of the water, is then

read directly from the tape. This measurement can usually be made by one person, even when the water is from 10 to 12 ft. deep.

In making a velocity measurement, the meter is held at the point in the stream at which it is desired to ascertain the velocity of the current. The wheel is allowed to revolve for a few seconds, in order that it may adjust itself to the current, after which the time for a given number of revolutions is noted, from which the velocity is obtained from the rating table for the meter. The time should be about 40 sec., and the number of revolutions will depend on the velocity of the water. A check on the work is made by repeating the observation. If the run is not repeated, a check can be obtained by noting mentally the time for each five revolutions. A stop-watch is used for observing the time, and the record is made to the nearest 0.2 sec., usually the smallest interval indicated with the ordinary stop-watch.

In discharge measurements, the mean horizontal velocity in a vertical at the measuring points is desired. Various methods are used for this determination, among which the following four are most common: (*a*) Vertical velocity-curve method, (*b*) two-tenths-and-eight-tenths-depth method, (*c*) six-tenths-depth method, and (*d*) subsurface method.

(*a*).—By the vertical velocity-curve method, measurements of horizontal velocity are usually made just beneath the surface, at 0.5 ft. below the surface, and at each fifth to each tenth of the depth from the surface to the bed of the stream, and as near the bottom as possible. These measured velocities, when plotted with depths as ordinates and velocities as abscissas, define for each vertical the vertical velocity-curve, which shows the velocity at every point in the vertical, and from which its mean velocity can be determined by dividing the area bounded by the curve and the axis of the ordinates by the mean depth. The following three methods are used in this determination:

1.—Determine the area with the planimeter and divide by the depth.

2.—Divide the area into any number of sections of equal depth, usually ten, and take the mean of the velocities at mid-point of each of these sections as the mean velocity.

3.—Divide the area into sections of convenient depth, which will be equal except for the bottom section, which may have an odd depth.

FIG. 1.—MEASUREMENT AT BRIDGE GAUGING STATION.

FIG. 2.—MEASUREMENT AT BOAT GAUGING STATION.

If the bottom section is the same depth as the others, take the mean of the middle ordinates of each section for the mean velocity. If the bottom section is an odd depth, multiply its mean velocity by the ratio of its depth to that of the common depth of the other sections, and add this product to the sum of the middle ordinates of the other sections, and divide by the number of sections.

Studies of vertical velocity-curves taken on many streams under various conditions of depth, velocity, and roughness of bed, show that these vertical velocity-curves have approximately the form of a parabola in which the axis, coinciding with the filament of maximum velocity, is parallel with the surface and is in general situated between the surface and one-third of the depth of the water. From the maximum the velocity decreases gradually upward to the surface and downward nearly to the bottom, where it changes more rapidly on account of the friction on the bed. As the depth and velocity increase, the curve approaches a vertical line as its limiting position.

The vertical velocity-curve method is valuable as a basis for the comparison of all other methods, for determining the coefficients to be used in reducing the values obtained by other methods to the true value, for use under new and unusual conditions of flow, and for measurements under ice.

(*b*).—In the two-tenths-eight-tenths method, the observations of velocity are taken at two points located at depths from the surface of 0.2 and 0.8 of the depth at the measuring point. The mean velocity is taken as the mean of the velocities at these two points. This method is based on the theory that the vertical velocity-curve is a parabola, as already stated, in which case the mean of the ordinates at 0.2114 and 0.7886 gives the mean ordinate. A study of a large number of vertical velocity-curves shows that this holds true in Nature; and experience proves that this method gives more consistent results than any of the others except the vertical velocity-curve method.

(*c*).—In the six-tenths method, the observation of the velocity is taken at a depth from the surface equal to 0.6 of the depth of the stream. This method is also based on the theory that the vertical velocity-curve is a parabola with the maximum abscissa between zero and one-third the depth, in which case the mean ordinate is between 0.58 and 0.67 of the depth from the surface. A study of a large number of vertical velocity-curve measurements shows that the mean depth

of the mean velocity is approximately 0.6 of the depth. This method has the advantage of ease in making the measurement, and gives very satisfactory results but not as good as those obtained by the two-tenths-eight-tenths method.

(*d*).—In the sub-surface method, the measurement of velocity is made at from 0.5 to 1 ft. below the surface, depending on the depth of the stream. The meter is held at sufficient depth to be out of any surface disturbance. When this method is used, the velocity must be reduced by a coefficient to obtain the mean velocity. This coefficient varies between 78 and 98%, depending on the depth and the velocity of the stream. The deeper the stream and the greater the velocity, the greater the coefficient. For average streams in moderate freshets, a coefficient of 90% should be used; in flood work, a coefficient of from 90 to 95%; and for streams at ordinary stages, from 85 to 90 per cent.

The results of a large number of vertical velocity-curve measurements, giving a comparison of the two-tenths-eight-tenths method, the six-tenths method, and the sub-surface method are given in Table 4.

Independent discharge measurements, as a rule, are of but little value in hydraulic work unless they are taken at stages which are known to be either extremely low or extremely high. In ordinary work it is necessary to make a series of measurements which, with daily gauge heights of the flow of the stream, enable the computation of the total flow of the stream and also its distribution. In connection with the individual measurements, therefore, it is necessary to observe gauge heights and take full notes of the conditions under which the measurement is made, in order to enable the construction of a station rating curve and estimate the daily discharge.

Computations of Current-Meter Discharge Measurements.

The computations of current-meter discharge measurements are usually made to determine: (*a*) the total area, (*b*) the mean velocity, and (*c*) the total discharge of the stream. The observed data from which these computations are made consist of (*a*) soundings at known intervals across the stream, (*b*) the mean velocity in the vertical at each sounding point, and (*c*) the distance between the points of measurement (Table 5).

The mean velocity, area, and discharge are computed independently

for each partial area included between perpendiculars from successive measuring points. The total discharge is the sum of the partial discharges. The computation of the partial discharge eliminates errors which would arise from the distribution of conditions existing in one part of the channel to parts to which they do not apply. The mean velocity is determined by dividing the total discharge by the total area of the cross-section.

The formulas used in connection with computations, may, in general, be classed as rectilinear and curvilinear—depending on the assumption that the bed of the stream and the horizontal velocity-curve vary from the measuring point either as a straight line or as a series of curved lines. A comparison of the computation of discharge measurements by various formulas, has been prepared by J. C. Stevens, Assoc. M. Am. Soc. C. E., District Engineer of the United States Geological Survey.*

In this discussion it is shown that the rectilinear formula:

$$D = l_1\left(\frac{d_0 + d_1}{2}\right)\left(\frac{v_0 + v_1}{2}\right) + l_2\left(\frac{d_1 + d_2}{2}\right)\left(\frac{v_1 + v_2}{2}\right) + \ldots\ldots\ldots\ldots\ldots l_n\left(\frac{d_{n-1} + d_n}{2}\right)\left(\frac{v_{n-1} + v_n}{2}\right)$$

gives the most accurate results and is readily used.

In this formula, d_0, d_1, d_2.....d_n and v_0, v_1, v_2.....v_n are the depths and velocities at the respective measuring points, a_0, a_1, a_2.....a_n, which are spaced at the distances, l_1, l_2, l_3.....l_n (Fig. 7). The area between any two perpendiculars at measuring points is equal to the mean of the depths at such points multiplied by the distance. Similarly, the mean velocities for the area between the two perpendiculars is equal to the mean of the velocities at the two perpendiculars. The product of this area and the mean velocity is equal to the discharge for the partial area between the two perpendiculars. Summing these partial areas and discharges gives the total area and total discharge. The mean velocity for the section is equal to the total discharge divided by the total area. Table 5 gives the field notes and computations for a typical current-meter measurement.

Low-Water Measurements.

At many stations the velocity of the river at low stages is so small that it may be advisable to find a section near by in which

Engineering News, June 25th, 1908.

Measurements Prior to 1905.											
Susquehanna	McCall Ferry, Pa	73	3.3–30.0	1.21–5.80	68		0.94		0.92	1.07	
Susquehanna	McCall Ferry, Pa	68	5.0–36.0	1.40–9.70	73		0.97		0.90		
Susquehanna	Harrisburg, Pa	20	3.2– 8.0	1.52–2.71	61		1.01		0.85	1.08	
Susquehanna	Binghamton, N. Y	36	2.5– 8.1	0.80–4.86	61		0.99			0.96	
Chenango	Binghamton, N. Y	34	1.7– 7.4	0.46–3.38	62		0.98			1.04	
Catskill Creek	South Cairo, N. Y	47	2.1– 7.0	0.27–3.91	58		1.02		0.83		
Delaware (E. Br.)	Hancock, N. Y	6	2.1– 4.8	0.75–4.12	59		1.02		0.82		
Delaware (W. Br.)	Hancock, N. Y	12	1.7– 4.2	1.67–4.45	58		1.02		0.83		
Esopus Creek	Kingston, N. Y	53	1.9–14.0	0.65–5.04	62		1.00		0.88		
Esopus Creek	Olivebridge, N. Y	68	0.9– 5.5	0.42–7.62	58		1.01		0.69		
Fishkill Creek	Glenham, N. Y	22	2.1– 5.0	0.26–1.75	59		1.04		0.79		
Housatonic	Gaylordsville, Conn	36	2.1– 6.2	0.67–3.66	58		1.01		0.83		
Rondout Creek	Rosendale, N. Y	21	4.4– 8.2	0.35–2.55	61		0.99		0.84		
Schoharie Creek	Prattsville, N. Y	11	1.1– 6.6	0.52–3.20	58		1.01		0.80		
Tenmile	Dover Plains, N. Y	33	1.8– 7.3	0.24–2.15	61		0.99		0.88		
Wallkill	New Paltz, N. Y	52	3.3–17.3	0.18–4.72	64		0.98		0.85		
Wappinger Creek	Wappinger Falls, N. Y	17	2.5– 4.7	0.58–2.11	58		1.03		0.78		
Appomattox	Mattoax, Va	8	2.5– 4.9	0.92–2.80	62		1.00		0.81	1.11	
James	Cartersville, Va	6	3.0– 4.4	1.88–2.37	60		1.01		0.88	1.08	
Roanoke	Roanoke, Va	8	1.8– 3.3	0.87–1.96	61		0.99		0.81	1.01	
Staunton	Randolph, Va	20	2.0– 6.0	1.35–3.11	66		0.96		0.95	1.12	
Dan	Madison, N. C	42	1.3– 3.8	0.73–3.00	62		0.99		0.86	1.01	
Dan	South Boston, Va	13	3.5– 9.0	0.90–2.41	64		0.98		0.90	1.09	
Reddie	North Wilkesboro, N. C	3	1.7– 2.5	1.62–2.01	58		1.02		0.83	0.97	
Yadkin	North Wilkesboro, N. C	33	2.7– 6.4	1.20–3.16	66		0.96		0.91	1.12	
Yadkin	Salisbury, N. C	10	3.1–11.3	0.96–3.23	59		1.02		0.81	1.06	
Catawba	Morgantown, N. C	39	1.9– 5.0	0.74–2.84	61		1.00		0.82	1.04	
Catawba	Mount Holly, N. C	2	6.3– 7.0	0.49–0.74	58		1.00		0.88	1.08	
Catawba	Belmont, N. C	1	3.5	1.54	59		1.01		0.80	1.05	
Wateree	Camden, S. C	4	12.3–17.7	1.33–1.83	58		1.01		0.90	1.13	
Broad	Alsten, S. C	4	5.0– 8.9	2.79–4.00	65		0.96		0.89	1.22	
Saluda	Waterloo, S. C	3	3.0– 8.5	1.76–2.01	62		0.97		0.82	1.03	
Watauga	Elizabethton, Tenn	10	2.6– 5.3	0.34–1.01	61		0.98		0.82	1.07	
South Fork Holston	Bluff City, Tenn	13	1.9– 4.1	1.08–6.77	62		1.02		0.80	1.05	
Nolichucky	Greeneville, Tenn	22	1.6– 5.1	0.49–2.69	59		1.02		0.80	1.02	
French Broad	Oldtown, Tenn	9	4.2– 8.0	0.53–2.18	62		0.99		0.86	1.07	
Pigeon	Newport, Tenn	4	4.4–11.6	0.82–1.14	61		0.99		0.91	1.15	
Tuckaseegee	Bryson City, N. C	36	3.5– 8.5	0.50–3.39	64		0.97		0.93	1.09	
Little Tennessee	Judson, N. C	11	2.7– 6.0	3.84–7.45	59		1.03		0.83	1.06	
Mean of 910 curves					61		1.00		0.85	1.07	
Highest			36.0	9.70	73		1.04		0.95	1.22	
Lowest			1.1	0.18	58		0.94		0.78	0.96	

* From "River Discharge."

TABLE 4.—Summary of Vertical Velocity Curves in Open Channels.*

Stream and locality.	Number of curves.	Range of depth, in feet.	Range of velocities, in feet per second.	Depth of thread of mean velocity, in percentage of total depth.	Depth of thread of maximum velocity, in percentage of total depth.	Coefficients for Reducing to Mean Velocity.					
						Six-tenths depth.	Mid-depth.	Top.	Mean of top and bottom.	$\frac{0.2+0.8}{2}$	$\frac{T+2M+B}{4}$
Measurements Since 1905.											
Ohio.....Wheeling, W. Va.....	14	5.3-20.8	0.92-4.43	66	13	0.98	.....	0.93	.....	1.005	.....
Monongahela.....Lock No. 4, Pa.....	7	10.9-17.5	0.74-1.13	68	0	0.96	.....	0.85	.....	0.990	.....
St. Mary.....Cardston, Alberta.....	8	2.7- 3.9	3.78-6.32	58	.....	1.01	.....	0.88	.....	1.001	.....
Jefferson.....Sappington, Mont.....	7	2.7- 5.7	1.78-3.45	63	.....	0.97	.....	0.84	.....	0.997	.....
Gallatin.....Logan, Mont.....	7	3.7- 5.5	0.88-3.85	62	.....	0.98	.....	0.85	.....	0.991	.....
Missouri.....Cascade, Mont.....	22	2.1-12.0	0.79-3.04	62	.....	0.99	.....	0.88	.....	1.002	.....
Marias.....Shelby, Mont.....	20	1.9- 4.2	0.57-4.52	60	.....	1.01	.....	0.86	.....	1.004	.....
Yellowstone.....Glendive, Mont.....	11	4.3- 8.5	2.42-3.79	60	.....	1.00	.....	0.89	.....	1.003	.....
Bighorn.....Fort Custer, Mont.....	8	3.9- 7.4	1.22-2.60	59	.....	1.00	.....	0.86	.....	0.998	.....
Shoshone.....Cody, Wyo.....	16	4.3- 7.3	4.93-8.45	58	13	1.02	0.96	0.87	.....	1.00	.....
North Platte.....Saratoga, Wyo.....	8	3.9- 5.1	1.82-3.61	63	26	0.98	.....	0.94	1.31	1.026	1.09
Pecos.....Dayton, N. Mex.....	10	4.1- 8.2	1.68-2.81	58	.....	1.03	.....	0.82	.....	1.003	.....
Pecos.....Pecos, Tex.....	6	2.2- 6.0	1.52-3.06	63	.....	0.97	.....	0.98	.....	1.004	.....
Green.....Greenriver, Wyo.....	36	2.2- 6.4	1.05-4.12	58	14	1.01	.....	0.90	1.20	1.026	1.06
Yampa.....Steamboat Springs, Colo....	8	4.2- 5.2	7.13-8.42	59	7	1.01	.....	0.83	1.10	1.003	1.02
Grand.....Hot Sulphur Springs, Colo..	6	2.8- 8.2	0.89-2.87	63	23	0.98	.....	0.85	1.30	1.01	1.03
Grand.....Kremmling, Colo.....	15	9.2-27.5	1.11-4.02	65	24	0.97	.....	0.93	1.16	1.001	1.03
Gunnison.....Cimarron, Colo.....	9	4.3-15.5	3.83-8.70	64	19	0.99	.....	0.92	1.06	1.003	1.02
Gunnison.....Cory, Colo.....	5	4.9-10.4	1.63-9.59	63	17	0.99	.....	0.92	1.09	1.007	1.01
Big Blackfoot.....Bonner, Mont.....	7	4.2- 6.8	1.41-2.49	62	.....	0.98	.....	0.85	.....	1.016	.....
Clark Fork.....Missoula, Mont.....	5	5.4- 7.6	1.72-2.41	60	10	1.00	.....	0.79	.....	0.997	.....
Yakima.....North Yakima, Wash.....	7	4.9- 7.6	1.58-2.56	59	.....	1.01	.....	0.83	1.06	0.999	1.00
Yakima.....Yakima, Wash.....	50	3.2-12.6	0.65-8.56	59	.....	1.01	.....	0.85	1.04	1.003	0.99
Yakima.....Kiona, Wash.....	20	2.0- 6.1	0.25-3.24	61	.....	0.98	.....	0.84	1.05	1.00	0.98
Kachess.....Easton, Wash.....	5	3.4- 4.3	1.44-4.78	61	.....	0.99	.....	0.85	1.04	1.009	0.99
Clealum.....Roslyn, Wash.....	20	4.7-11.7	0.37-3.89	58	.....	1.01	.....	0.84	1.09	0.995	1.02
Naches.....North Yakima, Wash.....	25	2.4- 6.0	0.65-7.38	58	.....	1.01	.....	0.85	1.04	0.998	0.99
Snake.....Minidoka, Idaho.....	14	4.0-10.7	1.13-3.05	62	20	0.99	.....	0.89	.....	0.994	.....
Big Lost.....Mackay, Idaho.....	7	2.4- 3.7	1.23-3.85	71	42	0.95	.....	0.95	.....	0.993	.....
Palouse.....Hooper, Wash.....	28	1.6- 8.0	0.97-6.49	60	.....	1.00	.....	0.84	1.06	1.004	1.00
John Day.....McDonald, Ore.....	15	3.6- 5.9	1.87-4.44	65	.....	0.98	.....	0.89	.....	.....	.....
Umatilla.....Pendleton, Ore.....	13	2.2- 7.5	0.47-2.73	64	.....	0.97	0.90	0.87	.....	0.970	.....
Deschutes.....Lava, Ore.....	10	3.0- 4.1	0.88-1.98	66	.....	0.99	0.94	0.92	.....	0.994	.....
West Fork Deschutes.Lava, Ore.....	27	1.7- 7.1	0.58-2.65	64	.....	0.98	0.96	0.89	.....	0.992	.....
Mean of 476 curves.....	.....	.....	.....	62	.....	0.99	.....	0.87	.....	1.001	.....
Highest.....	.....	27.5	9.59	71	.....	1.03	.....	0.98	.....	1.026	.....
Lowest.....	.....	1.6	0.25	58	.....	0.95	.....	0.79	.....	0.970	.....

TABLE 4.—*Continued.*

Stream and locality.	Number of curves.	Range of depth, in feet.	Range of velocities, in feet per second.	Depth of thread of mean velocity, in percentage of total depth.	Depth of thread of maximum velocity, in percentage of total depth.	Coefficients for Reducing to Mean Velocity.					
						Six-tenths depth.	Mid-Depth.	Top.	Mean of top and bottom.	$\frac{0.2+0.8}{2}$	$\frac{T+2M+B}{4}$
Shallow Streams with Rough Beds.											
St. Mary.....Dam Site, Mont.....	24	1.8– 4.2	2.60–6.36	60	9	1.00	0.93	0.86	.....	1.01	.....
Swiftcurrent Creek....Babb, Mont.....	16	1.6– 3.2	3.28–6.81	57	0	1.03	0.96	0.87	.....	1.004	.....
South Fork Shoshone.Marquette, Wyo.....	20	1.8– 3.6	3.61–6.79	58	0	1.02	0.93	0.82	.....	1.016	.....
Ana.....Summer Lake, Ore.....	6	1.8– 2.4	1.83–3.37	63	10	0.97	0.92	0.89	.....	1.00	.....
Tieton.....North Yakima, Wash.....	14	1.8– 4.5	4.42–8.04	56	0	1.04	0.96	0.80	.....	1.00	0.98
Wallowa.....Wallowa, Ore.....	4	2.2– 2.9	4.16–4.93	58	0	1.04	0.94	0.78	.....	1.004	.....
Kruzgamepa.....Salmon Lake, Alaska.....	108	1.6– 3.6	1.43–4.70	64	23	0.97	0.91	0.86	.....	1.004	0.99
Small streams.....Seward Peninsula, Alaska...	27	1.7– 2.5	2.15–3.47	61	14	1.00	0.93	0.84	.....	1.004	1.00
Mean of 219 curves.....	.....	.....	.....	60	7	1.01	0.92	0.84	.....	1.005	0.99
Highest.....	.....	4.5	8.04	64	23	1.04	0.96	0.89	.....	1.016	.....
Lowest.....	.....	1.6	1.43	56	0	0.97	0.91	0.78	.....	1.00	.....

TABLE 5.—Current-Meter Discharge Measurement Notes.

Date, Dec. 12, 1908, 9.15 A. M. Stream, Rock Cr. Party, F. F. Henshaw. Locality, Mass. Ave. Bridge, D. C. Meter No. 112. Gauge height, beg. 1.97, end 2.03, mean 2.00. Total area, 67. Mean velocity, 2.15. Discharge, 144.

Observations.					Computations.						
					Velocity.						
Distance from initial point.	Depth.	Depth of observation.	Time, in seconds.	Revolutions.	At point.	Mean in vertical.	Mean in section.	Mean depth.	Width.	Area.	Discharge.
0	0.0	.....	.....	0	.....	0.00					
							0.48	0.2	2	0.4	0.2
2	0.4	0.21	46	20	.....	0.96					
							1.22	0.5	2	1.0	1.2
4	0.6	0.36	44.4	30	.....	1.49					
							1.60	0.75	2	1.5	2.4
6	0.9	0.54	38.6	30	.....	1.71					
		0.23	42.4	50	2.63		1.88	1.0	4	4.0	7.5
10	1.1	0.92	44	30	1.50	2.06					
		0.3	31.4	40	2.84		2.16	1.3	2	2.6	5.6
12	1.5	1.2	39.2	30	1.68	2.26					
		0.36	39.8	50	2.79		2.24	1.65	2	3.3	7.4
14	1.8	1.44	39.6	30	1.67	2.23					
16	1.9	.....	.....	.....	.....	.....	2.28	1.9	4	7.6	17.3
		0.38	38.8	50	2.87						
18	1.9	1.52	36.8	30	1.80	2.34					
		0.4	36.6	50	3.05		2.42	1.95	4	7.8	18.9
22	2.0	1.6	34.2	30	1.94	2.50					
		0.38	38	50	2.93		2.49	1.95	4	7.8	19.4
26	1.9	1.52	43.8	40	2.02	2.48					
		0.36	39		2.85		2.57	1.85	4	7.4	19.0
30	1.8	1.44	45	50	2.47	2.66					
		0.32	45	50	2.47		2.35	1.7	4	6.8	16.0
34	1.6	1.28	40.8	30	1.62	2.04					
		0.26	32		2.08		1.95	1.45	4	5.8	11.3
38	1.3	1.04	40.4	30	1.63	1.86					
		0.24	49.4	50	2.25		1.90	1.25	2	2.5	4.8
40	1.2	0.96	40	30	1.65	1.95					
							1.88	1.15	2	2.3	4.3
42	1.1	0.66	36.4	30	.....	1.82					
							1.65	1.0	3	3.0	5.0
45	0.9	0.57	44.8	30	.....	1.48					
							1.36	0.9	3	2.7	3.7
48	0.9	0.54	35.2	20	.....	1.25					
							0.88	0.45	1	0.4	0.4
49	0.0	.....	.....	.....	.....	0.50					
									49	66.9	144.4

the meter measurement may be made by wading. In order to obtain a suitable section, it may be necessary to cut off part of the flow by damming the stream and modifying the channel so as to get sufficient depth and velocity for measuring.

For such measurements, the meter clamped on the rod (Plate IV) is the most satisfactory; in this the Price acoustic attachment is especially useful. In making a wading measurement, a tape line is

stretched across the stream in order to mark the measuring points. The engineer stands below this tape line and to one side of the meter (Fig. 1, Plate VIII) in order that he may not disturb the action of the meter. Iron rods, $\frac{3}{8}$ in. in diameter and 3 or 4 ft. long, and having a split at the top, may be used conveniently to support the tape when making such measurements.

Measurements Under Ice.

Measurements under ice should always be made by the vertical velocity-curve method, or by the two-tenths-eight-tenths method, the latter, by many comparisons with vertical velocity-curves, having been shown to give results closely approximating the mean velocity. In work of this class, if the water is not too deep and the velocity too great, the meter may best be held on the rod.

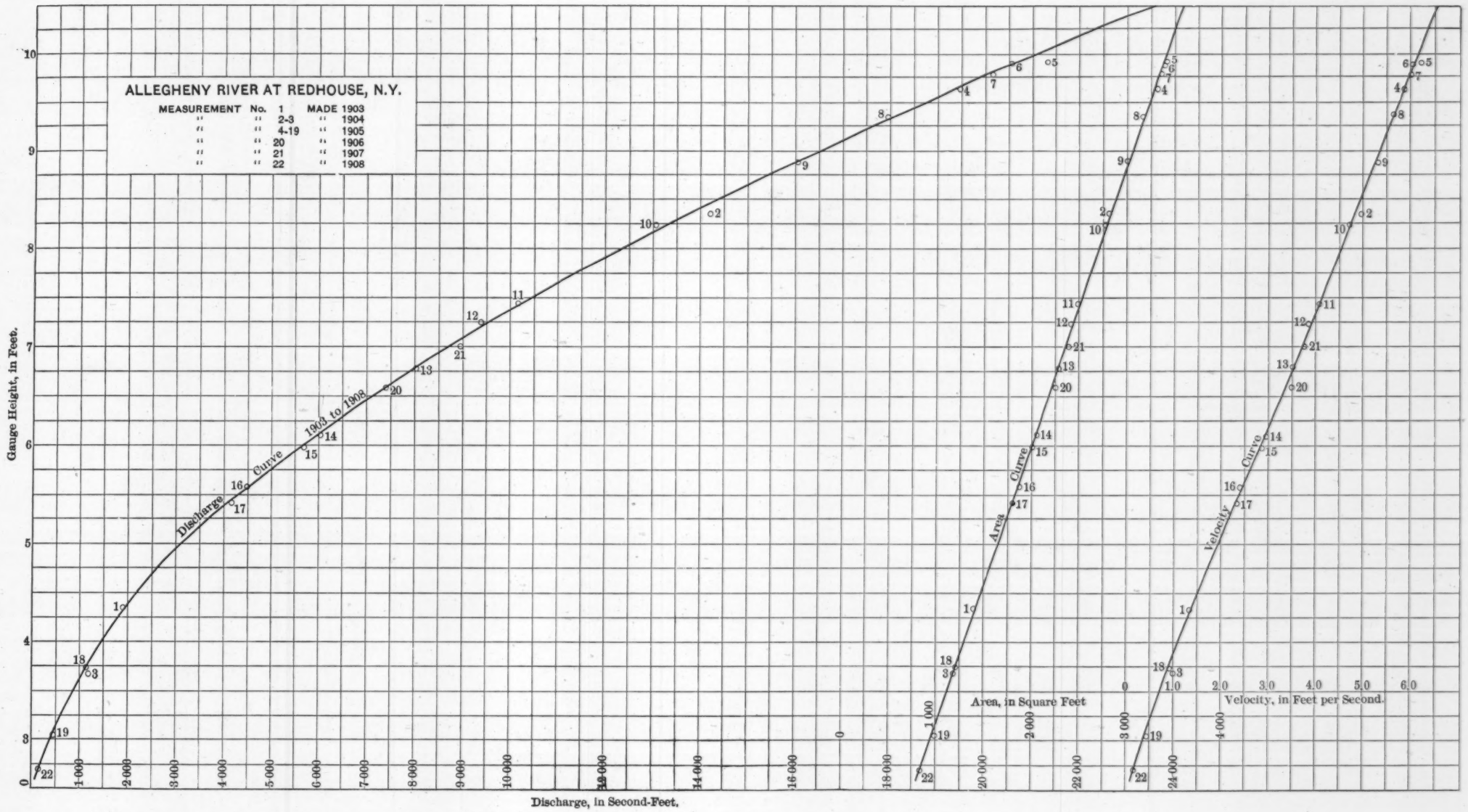
Measurements in Artificial Channels.

Current-meter measurement of the flow of water in artificial channels must be made with special care, as the laws of flow for open channels are not always applicable to artificial ones, because the water level may be subjected to disturbing influences such as undercurrents caused by intakes and outlets at rapid velocities.

Accuracy and Reliability of the Current Meter.

No attempt will be made herein to review the numerous discussions on the accuracy and use of the current meter. It is unfortunate that, in these discussions, most of the adverse arguments have been based on personal opinions rather than on the results of extended meter observations, and that from them there has resulted somewhat of a prejudice against the meter by those who are not familiar with its use and limitations. This prejudice, however, is rapidly disappearing, and, with increased use, the meter is coming to be recognized as a standard instrument in hydraulic investigations.

In the consideration of the accuracy of the results obtainable with the current meter, account should be taken of the use to be made of the data. It must be remembered that both the total flow of the stream and its distribution over the drainage area are constantly changing, and that the conditions existing at any time may probably never occur again. The flow which may be expected in any given stream, therefore, can be determined only by studying a series of


ALLEGHENY RIVER AT REDHOUSE, N.Y.
MEASUREMENT No. 1 MADE 1903
" " 2-3 " 1904
" " 4-19 " 1905
" " 20 " 1906
" " 21 " 1907
" " 22 " 1908
Discharge Curve 1903 to 1908.
Area Curve
Velocity Curve
Gauge Height, in Feet.
Discharge, in Second-Feet.
Area, in Square Feet
Velocity, in Feet per Second.

records extending over a long period; for this reason ultra-refined measurements would have no more practical value than records having a reasonable degree of accuracy.

As with most instruments, the accuracy and reliability of the current meter depend largely on the care taken in the measurement and the method used. In general, the errors which have been attributed to the meter have been due wholly to the fact that it was operated by men who were unfamiliar with it and with the conditions under which it could be used.

Its reliability is best shown by the results of the measurements taken at gauging stations on streams with permanent beds. These results show that measurements taken with different meters and by different operators at intervals throughout a series of years plot on a well-defined curve, Plate X.

An examination of the rating tables (Table 3) for various meters indicates that, barring a very slight variation due to slight differences in manufacture, a standard table could be used for all meters and still bring the results within the limit of error which should be expected in this work. The re-ratings of the meters, as used in the field, also indicate that they are not subject to great change, if they have not been damaged and if the pivots are kept in good condition. It is also noted that the meter is most reliable for velocities greater than 1 ft. per sec., which is the most common velocity in stream gauging.

A review of all the available information in regard to the accuracy of the meter, and of the data collected by the engineers of the United States Geological Survey during the last two decades, shows that the current meter is wholly reliable for general stream-gauging work, and that, in general, the errors due to the instrument when properly used will not exceed 5 per cent. It is believed that the results of the greater number of measurements are in error less than that amount.

DISCUSSION

Mr. Diamant.

ARTHUR H. DIAMANT, ASSOC. M. AM. SOC. C. E. (by letter).—Too much cannot be written on the value of current-meter gauging work, and Mr. Hoyt is to be commended for this paper. The writer has been in charge of gauging the New Croton Aqueduct of New York City for the past seven years, and thinks that a description of the gauging station and the method of making the gaugings may be of interest.

The station is located at Dunwoodie, N. Y., and is a circular shaft tapering from 13.5 ft. in diameter at the aqueduct to 6 ft. in diameter at the surface of the ground, with a height above the crown of about 10 ft. A portable frame house is placed over the opening for the protection of the observers.

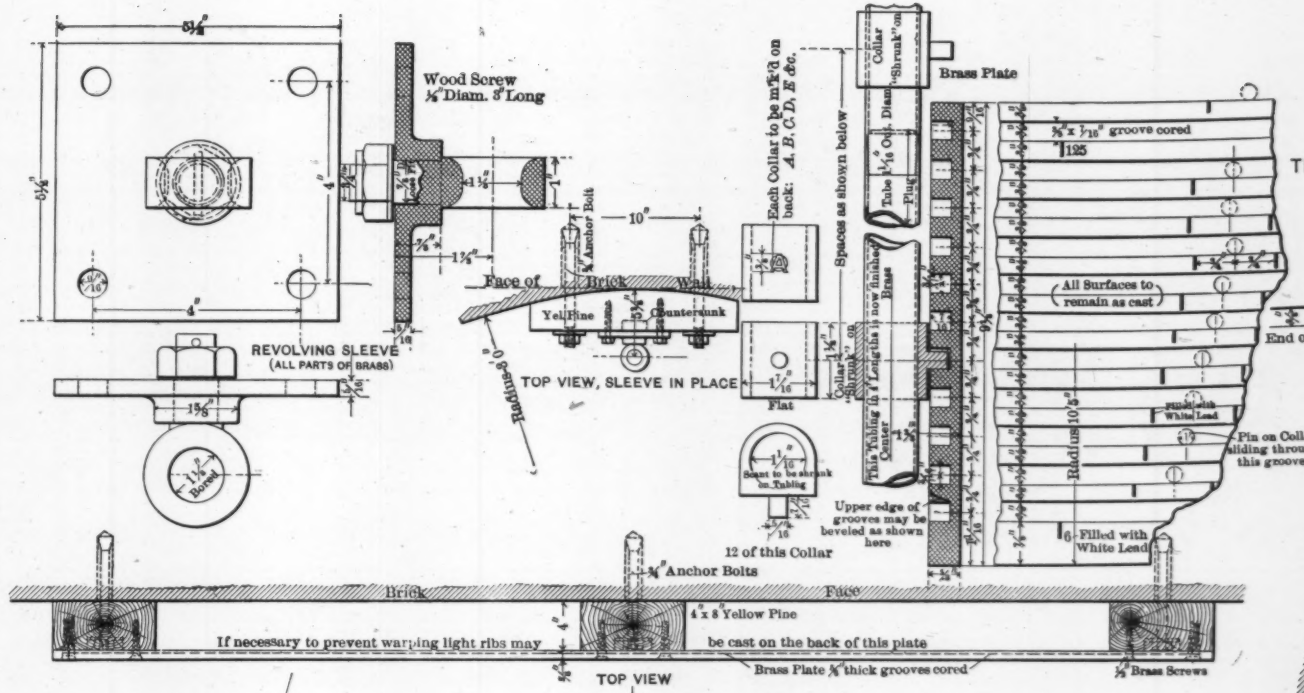
A bronze plate or limb, grooved and graduated as shown on Plate XI, is fastened to the side, and a revolving sleeve is attached near the top of the shaft for the purpose of holding and guiding the meter rod. This rod is a brass tube, $1\frac{1}{8}$ in. in outside diameter. It is fitted with collars and lugs, spaced as indicated on the drawing, and lettered to correspond with the grooves on the graduated limb. The area of cross-section of the aqueduct has been divided into areas of 1 sq. ft. each, and the revolving sleeve and graduated limb have been designed and placed so that the position of the meter is exactly known in each square foot of wet area. Plate XI shows a detail of the bronze plate, the rod and collars, the revolving sleeve, and also a general view of the gauging station with the limb in place and the meter in the first position. To prevent the rod from being bent down stream, a guide line (a cod-fish line) was attached about 1 ft. above the meter, the other end being fastened to a pole lying across a platform erected near the crown of the aqueduct for the men holding the meter rod in position.

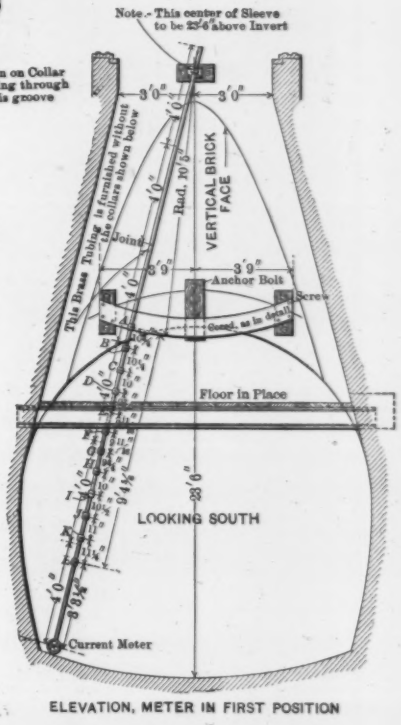
The current meter used is a Fteley and Stearns instrument, the wheel being 4 in. in diameter. During the past year a heavier instrument was used, the wheel being 5 in. in diameter. Before the writer took charge, the observations were read on dials geared to the wheel, the dial plate being thrown in and out of gear by a spring attachment. This method caused much loss of time, as the spring did not work satisfactorily when the meter was under 5 ft. of water, with a velocity of 2 or 3 ft. per sec., and as the meter had to be drawn out of the water after each observation. Very often the meter still recorded on being pulled up, thus spoiling the observation.

The writer after having used the instrument in this condition, recommended that an electrical recording device be attached, which was accordingly done, thereby cutting down the time of gauging the aqueduct to one-half. A small cam attached to the recording wheel strikes a brass spring at every revolution of the vane wheel, thus making and breaking the electrical circuit for each revolution of the meter.

GAUGING TEMPLET USED BY THE NEW YORK AQUEDUCT COMMISSION AT SHAFT No. 18

SECTION NO. 9

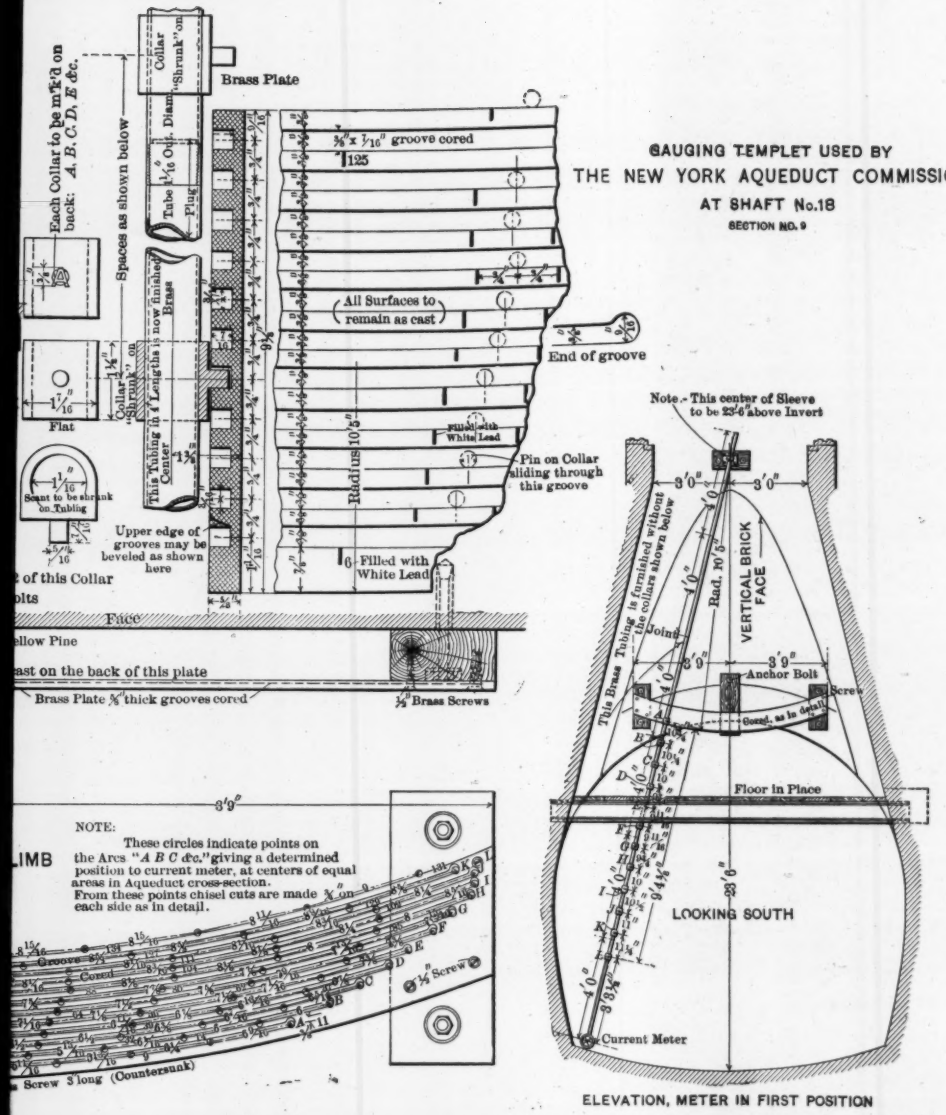




ELEVATION, METER IN FIRST POSITION

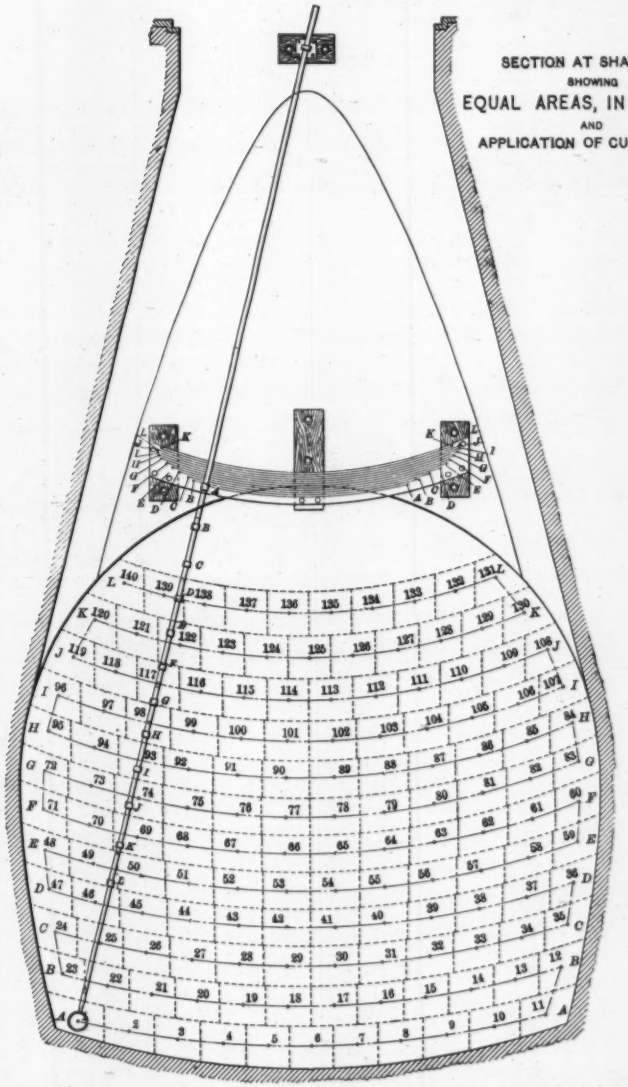
PLATE XI.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXVI, No. 1133.
DIAMANT ON
THE USE AND CARE OF THE CURRENT METER.

GAUGING TEMPLET USED BY
THE NEW YORK AQUEDUCT COMMISSION
AT SHAFT No. 18
SECTION NO. 9



ELEVATION, METER IN FIRST POSITION

SECTION AT SHAFT 18
SHOWING
EQUAL AREAS, IN AQUEDUCT
AND
APPLICATION OF CURRENT METER



Mr. Diamant.

An automatic counter recorded the revolutions when more than 100 per min., but for less than that number a sounder was thrown into the circuit, and the revolutions were actually counted. No trouble has been experienced, the whole apparatus working perfectly.

Current meters should be rated frequently, and as the Aqueduct Commissioners had no rating station, the meter had to be sent to Lawrence, Mass. As this occasioned much delay, a rating station was built at Dunwoodie, N. Y., on the writer's recommendation. A paved blow-off channel, 20 ft. wide and 600 ft. or more long, had been built in connection with the aqueduct, and, being in a valley, made an ideal place for the station. This consists of a timber dam with gates, for the purpose of flooding the channel to a depth of from 4 to 5 ft., a wooden platform 180 ft. long, carrying the track, and a car for holding the meter and rod. Fig. 1, Plate XII, shows the car, the current meter and rods, the electrical recorder and sounder, and the automatic switch which throws the recording mechanism in and out of the circuit. Fig. 2, Plate XII, shows the method of making the rating.

For a rating, the length of the course used was 150 ft., and about 50 observations were taken varying from about 0.5 to 8 ft. per sec. The car was pushed forward and backward over the course, to counteract any possible current. It was found that the friction increased so rapidly for velocities less than 0.5 ft. per sec. that the rate curve approximated a hyperbola. For a speed greater than 8 ft. per sec., the electrical recorder would not count properly. The method of taking observations was identical with that described by Mr. Hoyt.

The writer found that the ratings varied but little, and that for this Fteley meter, the rate curve was a straight line between the velocities, 1.2 and 8 ft. per sec., but for velocities of less than 1.2 ft. per sec. it was a hyperbola. Accordingly, in calculating the rate equation, he used the formulas $b \Sigma x + n a = \Sigma y$ for the straight line, and $a \Sigma x + b \Sigma x^2 = \Sigma x y$ for the curved part, where $x =$ the velocity, in feet per second, and $y =$ the number of revolutions per second. The observations were also plotted and a curve was drawn, which agreed very closely with the calculated curve.

Mr. Murphy.

E. C. MURPHY, M. AM. SOC. C. E. (by letter).—This valuable paper will be read with much interest by hydraulic engineers, especially by those who measure the discharge of streams or who consult the stream-gauging data of the Water Resources Branch of the United States Geological Survey.

The work being done by the engineers of this Branch, to simplify the current meter and to adapt it to the measurement of stream discharge, is certainly very commendable, and a comparison of their instruments and methods with those used in other countries cannot fail to show this fact.

The author's clear-cut description of the small Price meter and its

Mr. Murphy. accompaniments and his remarks on the care of the current meter are very good.

The writer is specially interested in the data of Table 3, showing the percentage of deviation of ratings of nine current meters from that of a mean rating of those meters. This comparison shows a close agreement in the behavior of meters of this type when in good condition and when operated in exactly the same way. In measuring the discharge of a stream, it is not always possible to have the current meter in exactly the condition it was when rated, nor to hold it in the water in the way it was held when rated, therefore a table showing the percentage of deviation of rating of a meter under various conditions which can be described accurately with a rating under standard conditions would be very valuable. With the data of such comparative ratings, an estimate can be made of the error due to conditions not being standard. The writer suggests that the meter be rated under the following conditions:

1.—With dull point,
2.—With one or more cups slightly damaged,
3.—With meter not well oiled,
4.—With water near freezing point,
5.—With weights near to cups,
6.—With cups close to bottom of channel,
7.—With cups close to water surface,
8.—With meter tipped upward or downward,
9.—With meter held with a rod not free to tip,
10.—With meter held with a rod free to tip.

The effect of a Pitot tube on the velocity of the water near the bottom of a channel has recently been observed by the writer in the U. S. Geological Survey Débris Investigation Laboratory. The channel was a trough with glass sides, carrying mixtures of water and sand. Although the part of the Pitot in the water had a width of less than $\frac{1}{8}$ in., as it approached the sand bed, the sand scoured from under it, leaving a space between the lower end of the Pitot of from $\frac{1}{4}$ to $\frac{1}{2}$ in., depending on the velocity. If a velocity-measuring instrument acts thus on the bed of a stream, the bed re-acts on the instrument and may affect its rating.

Mr. Barrows. H. K. Barrows, M. Am. Soc. C. E.—Several interesting questions have been suggested by this discussion. As far as the speaker knows, the rating of current meters in running water has never been attempted. While this experiment would be interesting, from a scientific viewpoint, it hardly seems necessary in a practical way, and the rating ordinarily made in still water is applicable to the ordinary case of stream measurements with no appreciable error. Further, comparisons of the results of current-meter measurements of discharge with the computed discharge over standard weirs, have been made in

FIG. 1.—CAR, CURRENT METER, RODS, ETC., NEW CROTON AQUEDUCT GAUGING STATION.

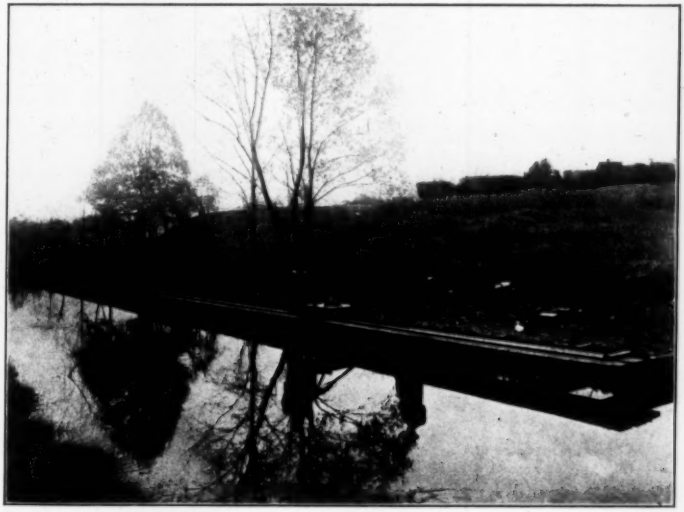


FIG. 2.—METHOD OF MAKING RATING, NEW CROTON AQUEDUCT GAUGING STATION.

a sufficient number of cases to establish the substantial accuracy of the current meter when rated as described in this paper. Mr. Barrows.

More investigation along the lines of practical hydraulics is needed, both in the use and improvement of such instruments as the current meter and in the derivation of suitable coefficients in other methods of estimating discharge. For example, in the use of gates to allow the discharge of stored water from lakes and ponds, little or no information is available on which to base the discharge under different conditions. To some extent the engineers of the United States Geological Survey are procuring information along such lines, but, with the limited funds available for stream gauging, it is obvious that little or no experimental work is possible. The speaker would urge more practical investigations on a full-sized scale by the universities and colleges equipped for hydraulic work, and the prompt publication of results.

The matter of coefficients for reducing surface velocity to the mean velocity in the vertical, as discussed by Mr. Grunsky, is important, as during the time of the year that, owing to the swiftness of the current, only surface velocities can be obtained, a considerable percentage of the total yearly run-off occurs, and errors in flood measurement may have serious results in the estimation of the quantity of water available for storage. This is one of the many problems of stream gauging studied by the U. S. Geological Survey, and for which satisfactory methods and coefficients have already been derived. With the limited appropriation for carrying on the work, however, and the uncertainty of the continuation of even this amount from one year to the next, it is not possible to maintain a full corps of competent men to carry on field work, so that some errors in results must be expected under present conditions. If a suitable appropriation were insured, extending over a period of at least ten years, many of the present sources of error could be avoided, and the work of stream gauging could be carried on continuously, in the broad and comprehensive way that it should be.

The speaker would urge especial care in the use of the current meter under such conditions as are met in head-bays, tail-races, etc., where the conditions of flow are governed by the manner in which water is being used, and may vary greatly during short intervals of time. The usual methods for the determination of mean velocity by current meter are not applicable, and, in general, the multiple-point or vertical-curve method is the only one which will give good results. The failure of the Price meter to give correct velocities where there are many cross-currents and much "boiling" must also be kept in mind, and a suitable section, reasonably free from such conditions, must be chosen for the observations.

The "two-tenths-and-eight-tenths-depth method," for the determination of the mean velocity in the vertical, has come to be the standard, both for open water and for ice cover. This has been of especial

Mr. Barrows.

interest to the speaker, as it was first suggested by him in 1905, as being a most promising method for the quick determination of mean velocity under all conditions.* This formula was developed in the study of vertical-velocity curves under ice cover,† and was found to apply even more truly to flow in open channels. It accords with the assumption that the typical vertical-velocity curve is a common parabola with horizontal axis. Some time after this formula was suggested by the speaker in 1905, he learned that, at a previous date, Thomas U. Taylor, M. Am. Soc. C. E., had concluded that, theoretically, the mean of observations at 0.21 and 0.79 depth should give the mean velocity in the vertical. The speaker would be interested to learn whether this method was ever in practical use prior to his adoption of it for work in New England.

An important point in respect to current-meter measurements under ice cover is that a single observation at 0.5 depth with the application of a coefficient ranging from 0.82 to 0.92, with a mean of 0.88, gives a quick and fairly reliable determination of the mean velocity in the vertical. This may be called the single-point method for ice cover, just as the six-tenths-depth method is for open water.

In concluding, the speaker wishes to emphasize the fact that criticisms and suggestions regarding the work and methods of the U. S. Geological Survey are welcomed at all times by its engineers, and that an effort is made to profit by them. The suggestions from practicing engineers, who make use of the estimates of stream flow, are especially valuable as, from their close study of these data, they realize just how improvement can be made.

Mr. Grunsky.

C. E. GRUNSKY, M. AM. SOC. C. E.—Under the direction of William Hammond Hall, M. Am. Soc. C. E., then State Engineer, many river gaugings were made in California in 1878, 1879, and the years following. The speaker was at that time an Assistant State Engineer in charge of the office computations of the gauging work, and took part in much of the field work.

The Sacramento, the San Joaquin, and the Feather Rivers were the principal streams in which the flow was measured. Both the current meter and floats were used in this work; but the results of gauging with floats were not dependable. The floats as used were on the same order as those with which the gauging work was done on the Mississippi River by Messrs. Humphreys and Abbot. The California work was a demonstration of the unsuitability of the sub-surface suspended float for use in stream gauging.

The current meters were of the Henry type. They were delicate instruments, each having two helical blades, about 1½ in. wide, attached to a slender spindle, the end of which rested in an agate bearing. A small piece of agate, let into a drum on the spindle, afforded the means

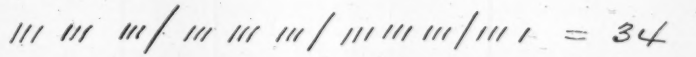
* *Journal of the Association of Engineering Societies*, July, 1905, pp. 19–20.

† Water Supply Paper No. 187, U. S. Geological Survey, pp. 84–85.

Mr. Grunsky.

of breaking an electric current which passed from the point of a light spring to the drum and spindle. An ordinary, but delicate, telegraphic sounder reported each revolution.

It was found that the counting of revolutions required sustained mental effort, and could not be kept up long without error, particularly when the observations extended through time periods of 3 min. Mr. Hall devised a system of marking down the revolutions with pencil strokes as follows:



This was a great improvement, and was very satisfactory, even at high velocities.

The data which accumulated during 1878 and 1879 were, some years later, made the basis of a special study by the speaker, and led to the preparation of a paper on stream gauging.* A brief reference to some of the conclusions then reached, and confirmed by later work, may be timely; but, just a word first about the stream-gauging work of the United States Geological Survey.

Whenever a stream or a drainage basin is to be studied, in order to determine its availability as a source of water supply for municipal or industrial purposes, for the development of power, for feeding a navigation canal, or for irrigation, more or less information relating to the ordinary stream flow, the range of flow, and the total yield of water in fixed time periods, must be secured. The feasibility of projects for the utilization of the run-off waters depends on this information. It is the same when drainage problems are under consideration. Where shall the engineer turn for the data? They cannot be secured in the field, at a moment's notice, by measuring rain or by measuring the stream flow. The stream to be investigated will rarely be found at such a stage that a short-time observation will furnish sufficient information to serve as a basis for reliable conclusions. Consequently, to answer with confidence the questions that relate to low-water flow, maximum discharge, or the aggregate output in a given time, he turns to the records, if there be any, for this or neighboring streams, and uses these records to amplify and to verify his own observations. Where there has been no measurement of the stream, or of other near-by streams, or of streams similarly circumstanced in the matter of physical features and climatic conditions, resort must be had to various methods of approximation.

It is self-evident that, the more complete the records are of stream flow and of rainfall in any section of the country, the greater will be the reliability of the conclusions reached, no matter whether they be based directly on long-time gauging records or only on rainfall records.

* Published as an Appendix in the Report of the Commissioner of Public Works, California, 1895, p. 77.

Mr. Grunsky.

Yet the importance of securing continuous records of stream flow has not been appreciated in the United States until within recent years, and even at the present time those who are insistent in demanding that such data be collected and published, even though in advance of the actual exploitation of water resources, are often suspected of having some ulterior motive behind their movement for the general good. No such motive is needed, however; the information wanted to-day should have been collected during the last 50 years, and the information collected to-day will be welcomed 50 and 100 years from now by the engineers who are then entrusted with hydraulic problems. Moreover, the work of collecting information relating to the water resources of the country and the rate of run-off that must be taken into account in planning the amelioration of flood conditions is never complete; it must be continued indefinitely, because the behaviors of the streams and of the drainage basins which yield water are as varied as the seasons themselves. We may wait for centuries for the extremes of the climatic conditions on which the extremes of river flow are dependent. These facts are known to the Engineering Profession, but the general public needs enlightenment.

The United States Geological Survey deserves commendation for the foresight which prompted the undertaking of such work by a scientific government bureau, and for the persistence with which the importance of this work has been urged upon the members of Congress, year after year. The published results of stream gaugings are of great value, and every break therein is to be deeply regretted.

The funds placed at the disposal of the Survey have been inadequate, and therefore the records, acceptable as they are, do not cover all parts of the country; neither has the lack of funds permitted the work to be done in all cases with the desired refinement. Moreover, the value of work of this character is appreciated by but a small portion of our population, and even limited annual appropriations are by no means assured.

It has been suggested—and it may be that the suggestion is a wise one—that the permanency of this work would be better assured if the hydrographic branch of the U. S. Geological Survey were made an independent bureau. In any event, it should be given authority (and ample funds) to make thorough and continuing investigations of the water resources in all parts of the country—East, West, North, and South. Full publication of information of this class will be of lasting benefit to the nation. To this fact engineers should call attention, as opportunity offers, for they are best qualified to testify to the loss that would be sustained by neglecting to collect and publish this information.

River discharge records, in all ordinary cases, are based on periodical gauging, with interpolation of the discharge for the time intervening between gaugings. The interpolation is generally made by aid of

Mr. Grunsky.

the hydrograph. It is essential, therefore, that at the gauging station the discharge be measured at varying stages of the stream, and, if the stream bed is not of a permanent character, that the gaugings be repeated frequently. The accuracy of the final record of discharge will depend on the appliances used in measuring cross-sectional areas and water velocities, on the accuracy of the hydrograph, as represented by the rod records, and on the skill and good judgment of the observer who makes the gaugings.

Each discharge measurement is based on a bank-to-bank determination of depth of water, and on a measurement of velocity at one or more points in the section. It is a well-known law that, in streams with beds of sand or gravel, the shape of the velocity-curve, if platted from bank to bank, is similar to the shape of the cross-section. In its position between banks the highest velocity is somewhere near the deepest water. It follows directly from this law that, to obtain highly accurate results, the velocity must be measured at a sufficient number of points from bank to bank to establish the shape of the horizontal curve, and that, where the depth is variable from bank to bank, single-point determinations of velocity, as for example the maximum surface velocity, will give results with a large probable error. Therefore it is not surprising that the experience of hydraulicians, the world over, has long ago led to the determination of the velocity at a number of points from bank to bank. As appliances have permitted, the attempt has been made, with more or less success, to measure not only the velocity at the surface, but to measure it at a number of points in the same vertical between the surface and the bottom, in order that the mean velocity in the vertical might be determined. It is evident that, if the mean velocity in a number of verticals is thus determined, the discharge through a vertical strip of the section having a width equal to unity would be: depth multiplied by velocity, $d\,v_m$, and the total discharge would be

$$D = \Sigma\, d\, v_m,$$

This is a curve easily constructed from the cross-section and the curve of mean velocities in the verticals, Fig. 8.

The substitution of the actual partial channel width, assigned to each vertical, for the strips having a width of unity leads directly to the method of computation which was adopted by the State Engineer of California in 1878.* The mean velocity in each vertical was applied to the portion of the cross-section lying between the median lines, separating the vertical from those next to it, or, in the case of an outside vertical, between each median line on one side and the river bank on the other. This method would be strictly correct if the verticals were close enough together, that is, if there were an infinite number.

A sample gauging, substantially in accord with the gauging of

* Report, Commissioner of Public Works, California, p. 98.

Mr. Grunsky.

Sacramento River at Gray and Shaws on February 17th, 1879, with a graphical computation of discharge, is shown on Fig. 8. The discharge is 39 120 sec-ft.

With the same position of verticals as shown in Fig. 8, the computation of discharge by the method adopted by the U. S. Geological Survey would be as shown in Table 6.

TABLE 6.—COMPUTATION OF DISCHARGE BY THE METHOD OF THE U. S. GEOLOGICAL SURVEY.

Gauging of Sacramento River, at Gray and Shaws by the State Engineer of California, February 17th, 1879.

Distance out, in feet.	Depth in feet.	Mean velocity in the vertical, in feet per second.	Partial products.
0	0	0	
65	10.8	2.15	$65 \times 10.8 \times 2.15 =$ 1 509
120	16.3	2.90	$55 \times 27.1 \times 5.05 =$ 7 527
170	16.8	3.15	$50 \times 33.1 \times 6.05 =$ 10 013
240	18.8	4.10	$70 \times 35.6 \times 7.25 =$ 18 067
310	21.5	4.40	$70 \times 40.3 \times 8.50 =$ 23 979
395	23.5	4.20	$85 \times 45.0 \times 8.60 =$ 32 895
465	25.5	3.75	$70 \times 49.0 \times 7.95 =$ 27 269
525	24.0	3.55	$60 \times 49.5 \times 7.30 =$ 21 681
596	0	0	$71 \times 24.0 \times 3.55 =$ 6 049
			$4\,D =$ 148 989
			The discharge, therefore = 37 250 sec-ft.

The computation by the method of the California State Engineer may be expressed by the following formula:

$$D = d_o v_1 \frac{l_1}{2} + d_1 v_1 \frac{l_1 + l_2}{2} + d_2 v_2 \frac{l_2 + l_3}{2} + \ldots\ldots$$
$$+ d_n v_n \frac{l_n + l_{n+1}}{2} + d_{n+1} v_n \frac{l_{n+1}}{2}.$$

The formula of the U. S. Geological Survey is mathematically as incorrect as it would be to use the area of the median section, instead of one-sixth of the sum of the end areas plus four times the area of the median section, in estimating the contents of prismoids.

The formula of the California State Engineer is practically the end-area formula, which is also an approximation. The one gives results that are too small, the other results that are too large. Either of these, however, is always sufficiently accurate when either depth or velocity is nearly uniform from bank to bank.

Mr. Grunsky.

STREAM GAUGING
THE GRAPHICAL METHOD OF DISCHARGE COMPUTATION

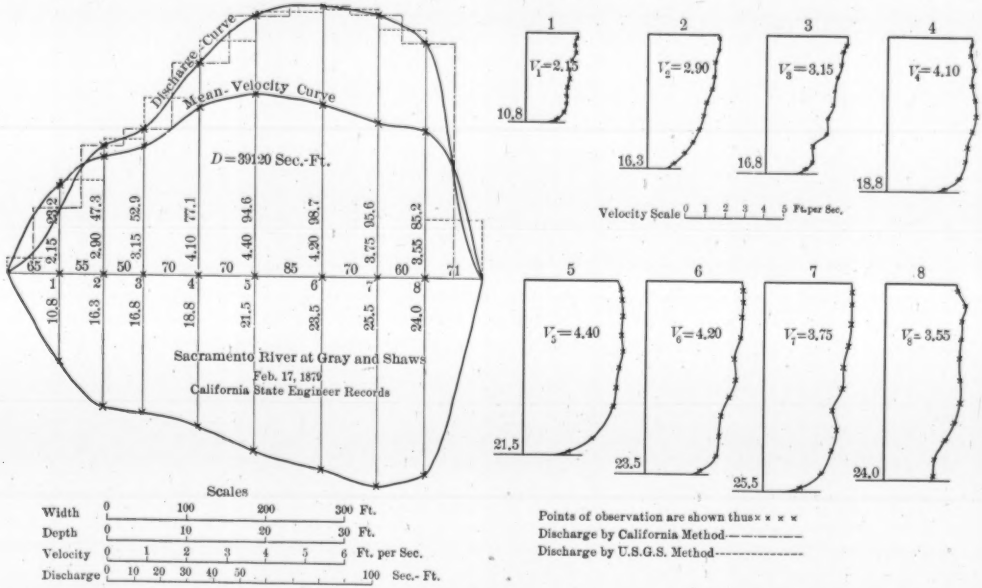


FIG. 8.

Mr. Grunsky.

The computation of a discharge by the California method is illustrated in Table 7. The values of both d_0 and d_{n+1} are zero in this case.

TABLE 7.—COMPUTATION OF DISCHARGE BY THE CALIFORNIA STATE ENGINEER'S METHOD (WILLIAM HAMMOND HALL, STATE ENGINEER IN 1879).

Gauging of Sacramento River, California, at Gray and Shaws, February 17th, 1879.

Distance out, in feet.	Depth, in feet.	Mean velocity in the vertical, in feet per second.	Partial discharge, in cubic feet per second.
0	0	0	
65	10.8	2.15	$(32.5 + 27.5) \times 10.8 \times 2.15 =$ 1 393
120	16.3	2.90	$(27.5 + 25.0) \times 16.3 \times 2.90 =$ 2 482
170	16.8	3.15	$(25.0 + 35.0) \times 16.8 \times 3.15 =$ 3 175
240	18.8	4.10	$(35.0 + 35.0) \times 18.8 \times 4.10 =$ 5 396
310	21.5	4.40	$(35.0 + 42.5) \times 21.5 \times 4.40 =$ 7 332
395	23.5	4.20	$(42.5 + 35.0) \times 23.5 \times 4.20 =$ 7 649
465	25.5	3.75	$(35.0 + 30.0) \times 25.5 \times 3.75 =$ 6 216
525	24.0	3.55	$(30.0 + 35.5) \times 24.0 \times 3.55 =$ 5 581
596	0	0	
			$D =$ 39 224 sec-ft.

It would be worth while to investigate thoroughly the superiority of the graphical method; but, in making the investigation, it should be remembered that the greatest variation in the results is to be expected in cross-sections with the greatest range of depth. The agreement will be best when the depth is nearly uniform from bank to bank. It is to be hoped that this investigation will be undertaken by the U. S. Geological Survey.

The example which has been cited gives:

By the graphical method	39 120	sec-ft.
" " California method	39 220	"
" " U. S. Geological Survey method	37 250	"

There seems to be no doubt that the graphical method of computation should be preferred whenever conditions permit. It is not convenient where the notebook field record is expected to show final results.

As long as there appears to be room for improvement in the appliances in use for measuring the velocity of flowing water, and for improvement in the methods of work, and in the interpretation of records, such papers as submitted by Mr. Hoyt are particularly acceptable to hydraulicians, and it is to be hoped that a way may be found to secure some device for measuring velocity, particularly in the larger rivers, which will be free from the objections that have been and are still being urged, to the current meter used for such work by

Mr. Grunsky.

the U. S. Geological Survey. While admitting the limitations of this device, the speaker does not wish to discourage its use—he is himself using one—but wishes to point out again, what has been done so often before, that there is room for improvement.

The Price current meter, to which these remarks refer, like nearly all meters with wheels, can be used under almost any conditions of turbidity, depth, and velocity, provided only that it can be lowered and held in the required position. Light driftwood, or floating leaves and blades of grass, are not likely to interfere with the free revolution of the cup-wheel. These are readily thrown off. If it be suspected that an observation has been affected by drift, the observation can readily be repeated. The Price current meter, properly used, is reliable whenever the set of the current is normal to the cross-section and the stream is free from eddies, whirls, and boils. It is used with a tail-vane, not for the purpose of holding a horizontal axis parallel with the direction of the current, but to keep those parts of the apparatus which can catch drift or which would disturb the flow of the approaching water in the rear of the wheel. It meets the requirement that the flowing water should reach the wheel and give it motion undisturbed by the supporting device; but, this meter has the defect that it responds to any horizonal current, no matter what its direction may be. The observer has no means of determining (when the current meter is out of sight) whether the velocity which the instrument reports is down stream or up stream or has some other direction. It is interpreted to be a down-stream velocity, and is so recorded. The cup-wheel, moreover, not only thus responds to every horizontal movement of the water, but the wheel revolves when there is motion parallel with its axis. The revolutions made under the influence of vertical currents are in the same direction as those caused by horizontal currents.* Therefore, whenever the meter is used to measure velocities in a stream in which there are eddies, whirls, and boils, it must be expected to give results in excess of the actual fact. This must be so, because the recorded number of revolutions is the result, not merely of the water movement normal to the section, but of the aggregate movement of water in a horizontal plane during the time of observation plus a possible acceleration by vertical currents.

As far as known, no experiments have ever been made to determine how great this error may be. The test measurements, as ordinarily made, throw no light on this question.

The screw-wheel class of current meters, among which the Haskell is probably the one best known to engineers in the United States, as they are ordinarily used in gauging rivers, are similarly defective. They are allowed to swing freely, guided by tail-vanes into the line of maximum horizontal current, and the velocity, as measured, is in all

* This is not from the speaker's personal observations, but seems to be well vouched for. See *Transactions*, Am. Soc. C. E., Vol. XLVII, p. 379.

Mr. Grunsky.

ordinary cases recorded as velocity normal to the gauging-section; but, in the case of meters of this class, the axis of revolution is horizontal and its direction can be ascertained by a direction indicator. It becomes possible to apply a suitable correction when the set of the current is not normal to the gauging section.

When allowed to swing freely into the line of the current, these meters, like those of the Price type, are caught by the eddies and whirls, and are turned into that direction in which there is for the moment the swiftest current. They report every cross-current and even reverse currents—unless used with a direction indicator—as though the flow were all normal to the section. Consequently, these meters also lead to an over-estimate of the discharge. How much any such meter is affected by vertical currents does not seem to have been ascertained by anyone. According to the method of construction, the direct effect of all water movement at right angles to the axis of revolution can be eliminated, as, for example, in the case of the Fteley meter, which has enclosed blades. It is not known whether or not the increased pressure upon the bearings, resulting from the velocity across the axis, modifies the rating of the meter.

Fortunately—no matter what the type of current meter—it seems to be reasonably certain that the errors of measurement resulting from cross-currents and from vertical currents are small when the gauging stations are located in long, straight reaches of fairly uniform cross-section, unless possibly when the current is disturbed by shifting sands on the river bottom; but, the error resulting from cross-currents can in large measure be eliminated if a current meter with a horizontal axis be used on a supporting device that will hold the meter axis normal to the gauging section. The best device for this purpose that has come to notice is that used by Mr. J. J. Revy in gauging the Parana and other South American rivers in 1874. He attached the current meter to the end of a horizontal bar which was lowered to any required depth by two supporting cords or wires.

The desirability of using the meter without a tail-vane was not recognized by Mr. Hall in the California work. He modified the Revy device by substituting a rectangle of light gas pipe* for the single rod. The meter could turn freely about the up-stream member of the rectangle. Of course, in the case of any current meter with a cup-wheel revolving in a horizontal plane, there would be only disadvantage in preventing the meter from swinging horizontally into the line of the current. It would be different if the meter were used with its axis horizontal in the plane of the gauging section. This, in fact, would appear to be the natural and best way to use the cup-wheel, particularly if a central disk be added, bisecting all the cups, to stop the movement of water through the wheel at right angles to the plane of revolution. No meter of this kind has ever been constructed. It

* Report, Commissioner of Public Works, California, 1895. Appendix.

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is worthy of a trial. A meter of this kind could not be used quite as near the water surface as some other meters. Moreover, in shallow water where, for every few inches in depth, there is considerable change of velocity, the meter would have top and bottom cups in water layers of materially different velocity. There would be undue acceleration if the open end of the cups were opposed to the swifter current and undue retardation in the other case. It is probable that this defect would not be material, because it can be offset, in whole or in part, by recording the velocity, not for the depth of the wheel axis, but for the depth below the water surface at which the upper or the lower cup, as the case may be, moves down stream in making its revolution.

Perhaps in the end this may be found to be a point in favor of and not against such a meter, because, by the use of two meters, one revolving (when looked at so that up stream is to the right) in the direction of the hands of a clock and the other in the reverse direction, it will be practicable, not only to eliminate any error resulting from the application of the measured velocity to the wrong depth, but one instrument can be used to measure velocity very close to the surface and the other velocity very close to the bottom. The use of two meters, however, would probably never be resorted to, except in cases where unusual accuracy of results is demanded.

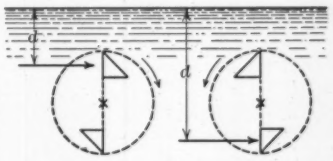

FIG. 9.

The method of lowering the current meter into position on a large river, as adopted by Mr. Hall in 1879, was so satisfactory that a brief description thereof may not be out of place. The river had a width of about 600 ft., maximum depths in the gauging sections of from 20 to 30 ft., and maximum surface velocities exceeding 6.5 ft. per sec. The current meter, on a rectangular frame, as already referred to, was lowered from a catamaran constructed by uniting two small skiffs with a light, easily detachable platform. The supporting wire cables of the gas-pipe frame passed over a drum which was turned by a crank. The frame was heavily loaded along the bottom member so that it could be used in measuring depths. The catamaran was held in place by a line to a boat anchored from 60 to 100 ft. up stream. From this boat a light wire cable extended to the top of the meter frame, which was thus prevented from swinging down stream out of the perpendicular. A steam launch shifted the upper boat, with the catamaran attached, from station to station.

A gauging, with velocity measurements, in ten stations or verticals, with 1 to 3-min. records for each tenth of the depth, can be made with such an outfit in a day. At each meter point, in the California work, the meter was held for 3 min.

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At times, during the work on the Sacramento River, much drift was running. This was a source of annoyance, not only because it made frequent inspection of the meter necessary, but because it was a menace to the anchored boat. On one occasion a large, flat-bottomed rowboat with a square bow was left unattended for half an hour, with the result that drift, for the most part the reed known as "tule," collected on the anchor line and swamped the boat. The boat in this case was lost. The swift current and the rapidity with which the boat filled with sediment, and above all the lack of suitable wrecking appliances, made its recovery impossible.

Although attention has been called to the errors that result from an improper interpretation of the wheel revolutions, it should be stated that these errors are ordinarily trifling when compared with those that may result from other causes. It is difficult to measure depth with accuracy in a deep, swift stream; but it is as necessary to measure depth, in all streams that have unstable beds, when gauging, as to measure velocity. Soundings in currents that are swift—6 ft. per sec. or greater—when the water is 20 ft. or more in depth may be 5% or more in error. The error results mainly from the fact that, at the moment when the sounding line is read, the lead may not be, as assumed, vertically below the observer; the line, moreover, is bowed more or less out of a straight-line position. Then, too, the line, if of hemp or cotton, may be of variable length between marks, according to temperature, degree of wetness, and the length of time that it is in use. Errors of the second kind can be minimized by a careful observer. The error of the first kind is more serious; it can be eliminated only by applying corrections based on experience. Such corrections are not applied in ordinary work. But this error, again, is in the same direction as the usual errors in measuring velocities. The deflected sounding line is too long, and the depths as recorded are greater than the actual depths. It is as important, therefore, to select proper devices for measuring water depths as it is to use properly designed and properly manipulated current meters.

How great the error may be, even in work done under intelligent direction, was observed at a gauging of the Colorado River, at Yuma, on June 24th, 1907. The presumption is that the gauging is typical of many other high-water gaugings at that point. The gauging was made by an observer who was detailed for the work from the hydrographic branch of the Geological Survey, and was carried at the time on the pay-roll of the Reclamation Service.

The observer was operating alone, from a car suspended from a wire cable stretched across the river about ¼ mile below the railroad bridge. The width of the river was about 600 ft., and the maximum depth of water was probably about 40 ft. It was recorded at 45.8 ft. The maximum surface velocity exceeded 7 ft. per sec. The soundings were made

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with a torpedo-shaped weight supported by a light wire cable. The stations were 30 ft. apart, and, at each station, the sounding weight was lowered first to the water surface and then to the bottom. The difference between the two readings was recorded as the depth of the river. The car, according to its position—in mid-stream or near the bank—was from 5 to 10 ft. above the water. The wire cable which supported the sounding weight passed over a small drum. The weight was too heavy to heave forward, and each sounding was made by lowering away. At the stations where the current was swift and the depth 30 ft. or more, the sounding line from the car to the water surface was inclined considerably. Angles exceeding 30° with the vertical were noted. No correction was applied for error from this source, not even for the part of the line above water. The recorded depths in the central two-thirds of the gauging section in this case may have been from 10 to 15% in error. The maximum depth of 45.8 ft., found by the observer, is probably about 6 ft. in excess of the actual depth at the time of gauging; but, the errors in depth were not the only ones. The observer calculated the lengths of line to be let out to lower the current meter to points at two-tenths and at eight-tenths of the depth. In the swift water off shore, the current meter did not go to these points. Near mid-stream, the supporting cord made an angle of about 30° with the water surface. The current meter, when supposed to be at one-fifth depth, was probably near the water surface; and, when supposed to be at four-fifths depth, it may have been anywhere between the surface and the supposed point, depending on the velocity. It was perhaps in most cases (near mid-stream) somewhere near the depth of maximum velocity. These statements are not based on the official records, but were noted independently.

It seems probable that, instead of the recorded discharge (in the main channel) of 101 200 sec-ft. on June 24th, 1907, the river carried only 86 300 sec-ft.*

The recorded discharge of the main channel (neglecting for the time being 11 200 sec-ft. additional, measured in a secondary channel) probably exceeds the actual on the day of gauging by about 17 per cent. To the recorded discharge of 101 200 sec-ft. a correction of 15% decrease is to be applied, in order to approximate the actual discharge. The total flow of the river for both channels, instead of being 112 400 sec-ft., as reported by the hydrographer, was about 97 500 sec-ft. The error here indicated is entirely independent of any error that may be due to the use of a Price current meter.

Hydraulicians have given much study to the shape of the vertical-velocity curve, that is, the curve which represents velocity from water

* This is the result of an estimate based on reduced depths and a reduction factor, 0.925, applied to the surface velocity to find the mean in the vertical. It was assumed that the velocities recorded for the one-fifth depths could be accepted as representing the surface velocities.

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surface to stream bottom. While there is much variation in the shape of individual curves, there is yet a general similarity that makes it possible to foretell its probable shape when information is at hand relating to the character of the stream and the width and depth of the flowing water. Using many vertical curves, as explained by Mr. Hoyt, a mean curve has been determined, by which a reduction factor can be obtained, and this factor, applied to a measured velocity at any selected depth, will give the probable mean velocity from surface to bottom—the mean velocity in the vertical. The points usually selected for such incomplete observations are at the surface, or at mid-depth, or at six-tenths of the depth, or, in combination, at two-tenths and eight-tenths of the depth. When streams of every possible character are to be gauged by one of these methods of approximation, the two-point method, or a measurement at six-tenths of the depth, or even at mid-depth, is to be preferred to the measurement of surface velocity alone. This is due to the fact that surface velocity has a relatively greater range of values, compared with the mean, than the velocities near mid-depth. Consequently, for quick work—conditions permitting—the two-point and one-point methods, in use by the U. S. Geological Survey, are good. As a rule, however, the conditions do not permit of the application of these methods when streams are at flood stage. Attempts to apply them result in such errors as have already been noted. Except when unusual facilities are provided, there is nothing that will give more reliable results for torrential streams in flood than the determination of surface velocities from bank to bank, from which, by the application of a proper reduction factor, the mean velocity in verticals, wherever located, can be approximated. This fact has long been recognized by the speaker, and an attempt has been made to ascertain the ratio to be expected between the surface velocity and the mean in typical cross-sections.* It was noted that the vertical curves are of similar types, in each cross-section, from bank to bank. It was also found that, for the same width of stream, the curves assumed a more erect position with increasing mean depth. Consequently, the following laws were formulated, and should be investigated further, on the basis of the mass of data now accumulated by the U. S. Geological Survey and other Governmental surveys.

1.—The ratio of the surface velocity to the mean velocity in any vertical is practically uniform from bank to bank in any cross-section of a stream.

2.—The surface velocity, in its relation to the mean velocity in any vertical, increases as the ratio of the width of the stream to the mean depth increases.

Furthermore, there appears to be some decrease of surface velocity, in its relation to the mean velocity in any vertical, when the depth

* Report, Commissioner of Public Works, California, 1895. Appendix.

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increases and when the mean velocity of the stream decreases. This matter, too, should receive further investigation, as also the effect, not only of the character of the river bottom, but also of the local river bed gradient.

The gaugings that led to the conclusions above stated were made in streams with alluvial, generally sandy, and therefore unstable, bottoms.

The ratios noted in Table 8 are probably a little low for inshore verticals and a little high for mid-stream verticals. They are intended to be applied to all surface velocities from bank to bank.

TABLE 8.—Reduction Factors to be Applied to Surface Velocity to Find the Mean Velocity in any Vertical.

Width divided by mean depth. $\frac{W}{d}$	Reduction factors. $\frac{V_m}{V_s}$	Width divided by mean depth. $\frac{W}{d}$	Reduction factors. $\frac{V_m}{V_s}$
5	1.01	40	0.87
10	0.97	50	0.85
15	0.94	75	0.83
20	0.92	100	0.82
30	0.89		

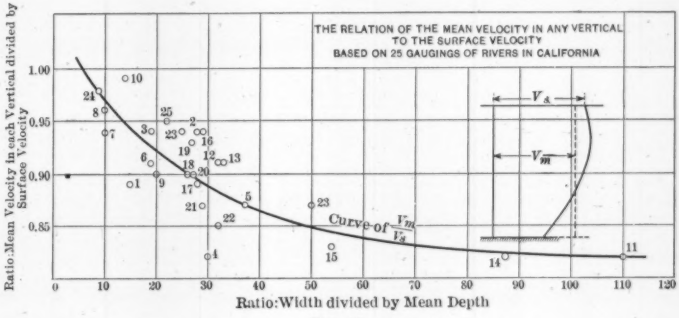

FIG. 10.

The curve on which these ratios are based is shown in Fig. 10.

The mean velocities, in the streams in which the gaugings furnished data for Fig. 10, ranged from about 1 to nearly 5 ft. per sec.

Further data relating to these gaugings are presented in Table 9.

For comparison, there are noted, in the columns headed *a*, *b*, and *c* of Table 9, three values of the ratio, $\frac{V_m}{V_s}$. Those in Column *a* are the arithmetical means of the values found for all the verticals of each

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gauging. Those in Column *b* are the ratios between the mean surface velocities from bank to bank and the corresponding means in all possible verticals. (These ratios were determined graphically by constructing for each gauging above the water line a curve of surface velocity and also the curve determined by the mean velocities in the several verticals. The area of the mean velocity curve divided by the area of the surface velocity curve is the value noted in Column *b*.)

TABLE 9.—GAUGINGS OF RIVERS IN CALIFORNIA BY THE STATE ENGINEER.

Number.	Date.			River and location.	Discharge.	Mean velocity.	$\frac{W}{d}$	$\frac{V_m}{V_s}$			Number of verticals.	Number of points in each vertical.
	Year.	Month.	Day.					*a*	*b*	*c*		
				Sacramento River.								
1	1879	4	6	Butte City.....	31 100	4.08	15	0.91	0.91	0.89	5	6 to 11
2	1878	8	2–5	Colusa (18).....	4 620	1.43	28	0.93	0.91	0.94	8	2 to 5
3	1879	4	4	Colusa (24).....	30 260	3.42	19	0.94	0.89	0.94	9	7 to 11
4	1878	8	13–14	Butte Slough.....	4 440	1.35	30	0.86	0.87	0.82	9	3 and 4
5	1878	8	15–17	Moons Ferry.....	5 020	2.06	37	0.87	0.87	0.87	11	3 and 4
6	1878	8	28–31	Knight's Ldg. (4)..	5 530	1.58	19	0.91	0.89	0.91	9	5 to 13
7	1879	2	18–19	Knight's Ldg. (4)..	18 180	3.13	10	0.97	1.00	0.94	6	10 to 20
8	1879	4	1	Knight's Ldg. (4)..	18 860	2.82	10	0.98	1.01	0.96	4	11
9	1878	8	31	Knight's Ldg. (29).	4 970	1.41	20	0.91	0.90	0.90	10	5
10	1878	9	20–23	Above Feather.....	5 430	0.95	14	0.99	0.99	0.99	10	4 to 9
11	1878	9	25–26	Below Feather.....	6 180	1.66	110	0.82	0.82	0.82	22	3
12	1879	2	13–15	Gray and Shaw.....	43 500	3.88	32	0.91	0.90	0.91	9	6 to 11
13	1879	2	17	Gray and Shaw.....	39 120	3.63	33	0.89	0.90	0.91	8	11
14	1878	11	7	Above American..	7 410	2.05	88	0.82	0.83	0.82	21	3
15	1879	1	31	Above American..	14 960	2.23	54	0.84	0.80	0.83	10	6
16	1879	3	10–11	Freeport.....	58 080	4.56	29	0.94	0.95	0.94	8	11
17	1879	3	14	Freeport.....	62 020	4.56	28	0.91	0.89	0.89	12	11
18	1879	3	17	Freeport.....	59 130	4.16	26	0.91	0.91	0.90	6	11
19	1879	3	18	Freeport.....	57 480	4.20	27	0.94	0.94	0.93	10	11
20	1879	3	19	Freeport.....	54 960	3.98	27	0.91	0.93	0.90	11	11
21	1879	3	28	Freeport.....	49 290	3.84	29	0.88	0.90	0.87	10	9 to 11
22	1879	5	26	Freeport.....	35 010	2.99	32	0.87	0.87	0.85	11	11
				Feather River.								
23	1878	9	10–11	At Mouth.....	1 720	2.53	25	0.94	....	(0.94)	4	3 and 4
23	1878	9	10–11	At Mouth.....	1 720	2.53	50	0.87	....	(0.87)	4	2
				San Joaquin River.								
24	1879	6	21	Above Paradise Cut	18 820	2.82	9	0.97	0.98	0.98	....	.....
25	1879	6	18	Johnsons Ferry....	.....	....	22	0.94	0.95	0.95	....	.....

NOTE.—In the case of gauging No. 23, the water was about 2 ft. deep for 100 ft. out from the left bank and from 3 to 6 ft. deep thence to the right bank. The ratio of width to mean depth is given separately for these two portions of the cross-section.

The values in Column *c* were found by giving double weight to the verticals near mid-stream, that is, to those within or close by the middle third of the width. It seems proper thus to give greater weight to mid-stream than to inshore verticals, in determining a ratio to be used for all, because both the greatest velocity and the greatest depth (therefore, too, the greatest discharge) are to be expected near mid-stream. The values in Column *c* alone were used in constructing the curve shown in Fig. 10.

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It will be seen that velocity measurements in more than 200 verticals were made the basis of the conclusion that the reduction factor to be used in finding the mean velocity from the surface velocity in verticals is not a constant. It would be worth while to have this study extended to the work that has been done on other rivers than those in California.

When the surface-velocity method of gauging is used, the discharge computation can be made with the measured surface velocities, either graphically or by the use of sub-areas, and the result is then to be suitably corrected by the application of the proper reduction factor.

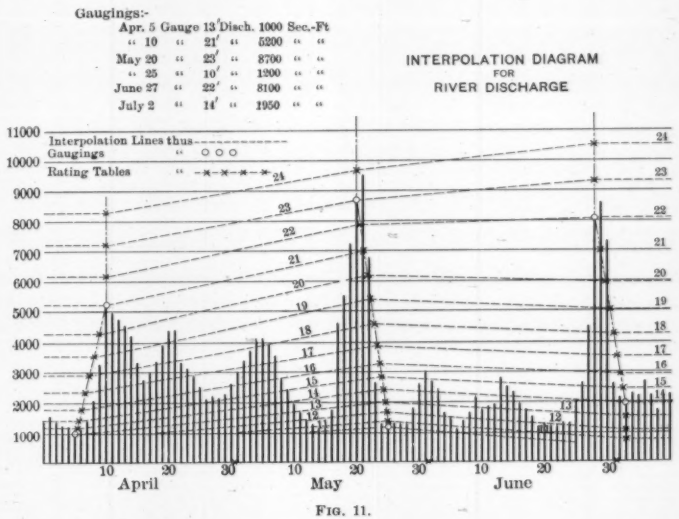

FIG. 11.

Passing over the preparation of a rating table, which is well illustrated by Mr. Hoyt's reference to the gauging work at Redhouse, it may be suggested that the interpretation of rod records on streams with unstable bottom is oftentimes quite as great a problem as the making of the gaugings on which the rating tables are based. It was found, for example, more than 25 years ago, that in years of excessive rainfall, such as 1861-62 and 1867-68, the Kern River, in California, builds up its bed at the Rio Bravo Ranch, where it breaks out of the mountains, and that thereafter for many years there is a progressive lowering of the river bottom. The low water of one year may be a foot or more below the low water of the preceding year. The interpretation of a

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gauge record, under these conditions, led to a method of discharge interpolation which may be found applicable under other similar circumstances. Gaugings were repeated at intervals to establish new rating curves. The change of the river bed was assumed to be gradual between gaugings. This led to the construction of a discharge curve substantially as shown in Fig. 11.

The vertical scale is the discharge, in cubic feet per second. The rating tables, as determined at such dates as April 5th, April 10th, May 20th, May 25th, June 27th, and July 2d, established points for the foot marks on the gauge rod, and a connection of these foot-mark points gave a series of interpolation lines, such that, for the rod reading on any date, as found on the diagram by these interpolation lines, the discharge could be read off. To complete the diagram, the lines representing discharge on each day were drawn up from the base line to the proper gauge height. All interpolation is thus made graphically. The daily discharge can now be read off, or, for stated periods, it can be determined by planimeter. The diagram presented is a sample, and has no relation to Kern River. The originals of the Kern River work have not been accessible.

Mr. Kuichling.

E. KUICHLING, M. AM. SOC. C. E.—This paper is a valuable addition to the literature of current meters, and brings to notice some interesting details of construction and calibration which are worthy of the closest study. In regard to construction, the aim is to reduce all frictional resistances to a minimum, so as to make the instrument as sensitive as possible. Although this seems to have been accomplished in high degree in the small Price meter, there is still room for improvement, as it is found that the apparatus does not give reliable indications when the current is less than 0.50 ft. per sec. The reason for this limitation is not apparent, and doubtless an explanation would be read with interest.

With reference to the method of calibrating the instrument, some further particulars are desirable. It has been found that when the meter is fixed to a rod and moved through still water at uniform velocity, the number of revolutions per minute made by the wheel is greater than when the meter is attached to a thin cable and heavy sinker, as in actual practice in river gaugings. This fact is pointed out by Edward C. Murphy,* M. Am. Soc. C. E., who states:

"That if the rating table for this meter held with a cable be used to reduce observations made with the meter when held with a rigid rod, the results will be in error from 1.3 per cent. to 8.5 per cent. for velocities from 1.5 feet to 0.5 foot per second."

This error may not be large, in the great majority of cases where current meters are used, yet it is important to have a clear knowledge of the limitations of the instrument, in order that suitable corrections

* Water Supply and Irrigation Paper No. 95, U. S. Geological Survey, 1904, pp. 87-89.

can be made when it is used in cases where a higher degree of accuracy is desired. Mr. Kuichling.

It was also found in Mr. Murphy's experiments at Cornell University, in 1901, that there was some difference whether the meter was calibrated in still water or in running water. The surface velocity of the running water was measured with 6-in. cubes of wood loaded with lead so as to have immersion of 5.25 in., and the Price meter was held with its center about 3 in. below the surface. In 36 observations, with a nearly constant velocity of 3.076 ft. per sec. by the floats, the meter indicated a velocity of only 2.806 ft. per sec. when held with a cable.

"It was thought that possibly this failure of the small Price meter to give correct surface-velocity indications was due to wave motion of the meter. Another experiment was therefore made with the floats and with the meter held with a rod and not free to tip. * * *"

The mean surface velocity shown by the small Price meter from 32 observations was 1.967 ft. and the corresponding float velocity was 2.146 ft. per sec. These figures indicate that in both cases the meter gives only about 91% of the float velocity. On lowering the center of the meter from 2.75 to 4.50 in. below the surface of the water, the velocity given by the meter increased from 1.884 to 2.035 ft., while the float velocity was 2.137 ft. per sec.

From these and similar experiments, Mr. Murphy concludes:

"That a small Price meter will not measure velocity correctly when its center is within 0.5 foot of the surface, if the velocity be greater than 1.5 feet per second, and that this error increases from 0 at about 0.5 foot below the surface to 8 or 9 per cent. at 2.5 inches below the surface."

It may be remarked, however, that the inference of correct meter indication at a depth of 0.5 ft. below the surface is not apparent from the context, as the true velocity at that depth does not appear to have been observed. In view of the fact that the velocity of the water usually increases from the surface downward to a depth of a foot or more in a canal of uniform rectangular section and 6 ft. depth, it is very probable that the velocity at the bottom of the 6-in. cubes was appreciably greater than that of the cubes, which is the resultant of the varying velocities from the surface to a depth of 5.25 in.

This consideration of calibrating a current meter brings up the question of how to measure accurately the velocity of running water at any point in the cross-section of a stream. The thing sought is the resultant velocity normal to the cross-section at the given point, while the actual current at this point may form a large angle with such normal direction, either permanently or temporarily. In most cases the direction of the longitudinal axis of the current meter, when held with a cable, is not known at a depth of more than 2 or 3 ft., so that

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a correction in this respect is impracticable; and, in deep streams, it is equally impracticable to hold the meter rigidly on a rod, without inducing disturbances of some kind, either in the water or by vibrations in the rod and meter.

The best practical test of the general accuracy of meter observations is the comparison of the computed discharge in the entire cross-section with the discharge measured by a standard weir. Thus far, however, this comparison has been applied only to a few cases of canals having a uniform rectangular cross-section, and the question arises as to the results in natural channels with various degrees of roughness or irregularity of bottom. To the speaker it seems to be very desirable that such experiments shall be undertaken by the U. S. Geological Survey, as the means for such work are rarely available to individuals or communities.

It is commonly assumed that the impact of running water against a stationary object immersed therein is exactly the same as the resistance to the motion of the same object at equal velocity through still water. This assumption has been investigated experimentally by Fink,* with the result that some difference was found when the velocity exceeded certain limits, also when the objects had different forms and lengths. Presumably, such difference was due to the action of the water on the rear surface of the immersed object by the formation of eddies of more or less intensity, but an entirely satisfactory explanation of the matter was not given. The subject is also considered briefly in Dr. M. Schmidt's valuable paper entitled "Investigations of the Rotary Motion of Current Meters."†

These remarks are not made with any intent to disparage the excellent work done by the U. S. Geological Survey, but solely to elicit from the author some additional facts in relation to calibrating accurately a current meter of the Price type which may be useful to practicing engineers. Such information may be available in various other publications, but it should certainly be incorporated in this interesting paper.

Mr. Matthes.

GERARD H. MATTHES, M. AM. SOC. C. E. (by letter).—The writer would like to emphasize the following paragraph in Mr. Hoyt's paper:

"As with most instruments, the accuracy and reliability of the current meter depend largely on the care taken in the measurement and the method used. In general, the errors which have been attributed to the meter have been due wholly to the fact that it has been operated by men who were unfamiliar with it and the conditions under which it could be used."

There is, in some quarters, a belief that the current meter is not as accurate an instrument as it ought to be. Mr. Hoyt's statement is true

* *Civilingenieur*, 1892, pp. 539 and 653 *et seq.*

† Forschungsarbeiten auf dem Gebiete des Ingenieurwesens, Berlin, 1903, No. 11, published by the Society of German Engineers.

of any instrument. An inexperienced person could not be expected to get first-class results, for instance, with a theodolite on triangulation. This work requires great skill; so does current-meter work. Mr. Matthes.

Another point of interest is that, in gauging streams, the current meter, in the hands of a well-trained observer, gives results of a higher degree of precision than can be obtained with the other data which are required for making up stream records. These other data relate to the stage of the stream, and are chiefly in the form of gauge height readings. Ordinarily, the stage of a river is read once or twice a day; in a few instances, known to the writer, it is read as often as once an hour, or is obtained with a self-recording instrument. As a general thing, the parties interested, for reasons of economy, must be satisfied with reading the river gauge once or twice a day, and, on the basis of these observations, such readings must form the records of the discharge of the river.

The fluctuations which take place in a natural stream are incessant, and, usually, are such that one or two isolated gauge readings in one day cannot give a correct estimate of the average discharge for that day. Some Colorado mountain streams, equipped with self-recording water-stage registers, show a periodic fluctuation between noon and midnight amounting to as much as 100 sec-ft., when the total discharge does not exceed 300 or 400 sec-ft. At the best, isolated gauge readings taken on such a stream, would form a basis for a rough approximation of the stream's discharge.

In short, the relative precisions obtained from ordinary gauge readings, and from current-meter work, are not at all commensurate, the precision of the current-meter work being far ahead of that derived from the other observations.

D. G. Thomas, M. Am. Soc. C. E. (by letter).—During the past year the writer has been experimenting with an automatic registering device for measuring the flow of water in a running stream. The apparatus is designed to eliminate the weaknesses of other devices which have failed in this work because of freezing in winter. Under test the apparatus registers automatically upon a dial in the same manner as steam and electric currents, so that the fluctuations in stream flow are closely recorded. Its indication depends on the water head above a broad weir, the head of water exerting a pressure on the bed of the weir, which, in turn, is communicated to a tube of very small diameter filled with air. This tube communicates with a piston which, as the dial revolves, actuates the traverse of the recording needle across the face of the dial. The device accommodates itself quite elastically to changes of temperature, and the air-filled tube is not subjected to freezing like water-filled tubes in similar devices. The real controlling device as used is submerged to a sufficient depth to prevent ice from forming over the weir. Mr. Thomas.

Mr. Thomas. The winter of 1908-09 was an unusually severe one for Denver, and, although the thermometer did not go extremely low, the cold was uniform. Under these conditions the apparatus recorded accurately, as checked by a mechanical device kept in a warm room for the purpose of comparison. It is intended to place the instrument at Lake Cheesman, where, at times, it is colder than in Denver, for the purpose of determining whether it will be successful in obtaining a full year's record under the varying temperatures there.

Mr. Pearl. WALTER PEARL, M. AM. SOC. C. E. (by letter).—The precision of the current meter is like that of the engineer's transit, the instrument itself is all right, but, sometimes, in checking up the work, it is found that errors have been made by the chainmen.

In ordinary gauging work, the cross-sections of streams are not taken with sufficient accuracy. In ordinary practice, meter calculations are made to three or four decimal places, but, in many cases, the cross-sections have been taken at intervals of 10 ft. or more. More care should be exercised in taking cross-sections. The instrument, if carefully rated, will give accurate results.

With its anti-friction adjustments and appliances, the current meter, as now constructed, is a very refined instrument, and its mechanical errors are hardly appreciable. It is a great and necessary aid to the hydraulic engineer, and the results obtained by its use should be considered as accurate information for the purposes required.

Mr. Allen. C. M. ALLEN, ASSOC. M. AM. SOC. C. E. (by letter).—Mr. Hoyt's paper is especially interesting to the writer because, during the past few years, he has often had occasion to use current meters for measuring the discharge of water-wheels after installation, and also in checking up the discharge, as measured by the current meter, when the water was also measured over a standard weir. About two years ago, a circular current-meter rating station was installed at the Hydraulic Testing Laboratory of the Worcester Polytechnic Institute. This station is located in the immediate vicinity of the Institute, in a pond of still water having an average depth of from 7 to 8 ft. It consists of a beam rotating about a vertical steel shaft, which is embedded in a large boulder conveniently situated in deep water about 50 ft. from the shore. The two arms of the beam are 42 and 21 ft. long, respectively, and the beam is trussed and suspended on a roller-bearing, being supported by a 6-in. pipe flanged at both ends, on the upper end of which is a 4-ft. rope sheave. The steel shaft is guyed in three directions with steel cables and turnbuckles in order to keep it rigid and plumb.

Just below the gates in the spillway of the pond there is a 24-in. vertical Hercules turbine acting under a head of about 6 ft., which furnishes power to rotate the beam by using a worm-gear and a rope drive. The current meter to be rated is attached to the beam at a known distance from the center, and the vanes are submerged to any

Mr. Allen.

desired depth. The wheel being started, the beam swings around over the surface of the water at a constant velocity, due to the facts that there is a constant load on the wheel and that it operates under a constant head. By opening or closing the gates of the turbine, the rotative speed of the beam is affected; also, the meter can be placed at any distance from the vertical shaft, and thus the velocity of the meter through the water can be varied from somewhat less than ½ ft. per sec. up to 12 ft. per sec. This range in velocity is considered sufficient to cover the majority of current-meter measurements.

When the current meter is attached to the outer end of the beam, one revolution causes it to travel more than 260 ft. Any disturbance in the water, caused by the passage of the current meter, is very quickly dissipated in the eddy currents thrown out in all directions; long before the beam has completed one revolution—even when the meter is traveling at 12 ft. per sec.—the water through which the meter has just passed has become quiet.

Experiments on rating current meters have been made in laboratories, the principal difference being that, in the laboratory, it is necessary to use a circular trough, and it has been discovered that, after the current meter has been around once or twice, the water is inclined to follow it, but this tendency is practically done away with by using a deep-water pond. A large number of tests which have been made at the Institute rating station show remarkably consistent results. Apparently, it makes very little difference whether the meter is attached to the end of the beam or within 10 ft. of the center, provided the velocity is not too great when the radius is small.

The Price current meter owned by the Institute has been rated several hundred times with this apparatus, and its rating has been found to remain practically constant, being now well within 1% of the maker's rating sent with the instrument. The Institute rating consisted in moving the meter through still water with velocities ranging from ½ ft. to 11 ft. per sec., clamping the meter on the beam at different distances from the center. The length of the radius seems to have no appreciable effect on the rating, within a wide range. With a radius of 12 ft., the meter was rated from less than ½ ft. up to a little more than 3 ft. per sec.; with a radius of 29 ft., it was rated from about 1½ up to 7 ft. per sec.; with a radius of 42 ft., it was rated from 4½ up to 11 ft. per sec.—the points from all these tests coming practically on a curve plotted from the maker's rating.

Several times tests have been made where water, going through a stone culvert very similar to the tail-race of a water-wheel plant, was measured by using meters of two or three different types, all of which had first been rated at the circular station. The water coming through this culvert was measured also over standard weirs with end contractions. The maximum error in any of six complete tests was about 3%, while the minimum was about ½%, the average being within 2 per cent.

Mr. Allen. The current meter is like many other hydraulic instruments for measuring water, in that it must be used carefully and properly. The Price meter will read large in measurements of tail-race discharge where there are cross-currents, as it is bound to pick up and register the swiftest velocity, regardless of the direction in which it comes. For conditions similar to those just mentioned, the Fteley meter will give more accurate results.

Mr. Hoyt, in his rules for the use and care of current meters, has certainly covered the ground well, and, although the writer believes that one cannot be too particular in the treatment of such instruments when making tests, he is forced to admit that the Price current meter can stand the ordinary handling of students for several years and still rate and measure within 2% of ordinary engineering requirements.

Mr. Hoyt. JOHN C. HOYT, M. AM. SOC. C. E. (by letter).—In reviewing the foregoing discussion, the writer wishes to emphasize the point brought out by Mr. Matthes. "The current meter, in the hands of a well-trained observer, gives results of a higher degree of precision than can be obtained with the other data which are required for making up stream records. These other data relate to the stage of the stream, and are chiefly in the form of gauge height readings."

The changes in the gauge height during the day, due to temperature or to artificial control, make it impossible on many streams to determine with any accuracy the mean daily stage from one or two readings a day. Most streams in the East are now largely affected by dams at power plants, which in many cases practically cut off all flow except during the operation of the plant. The temperature effects are chiefly on streams fed by snow in the mountains.

The need of obtaining continuous gauge readings with an automatic gauge is now being fully investigated by the Water Resources Branch of the United States Geological Survey, and plans for next season's work contemplate the installation of a number of these gauges on the more important streams. The maintenance of such automatic gauges as are now available is difficult, but it is expected that a gauge will soon be developed which will meet the conditions.

Mr. Grunsky's discussion is of special interest in that it outlines the methods used by others than the United States Geological Survey, and points out new lines of investigation. It was hoped that more discussion of this nature would be presented. A report has just been received from the Survey Department of Egypt on "Measurement of the Volumes of Discharge by the Nile during 1905-06," which outlines methods very similar to those used by the United States Geological Survey. The large Price meter has been used in this work, and satisfactory results have been obtained.

Referring to the method of computing the discharge measurements, the results will depend on the refinement used in making the observa-

tions. In a carefully made measurement the points of observation should be taken close enough together to determine any sudden change in depth or velocity. For small streams the distance between measuring points varies from 2 to 5 ft.; on the larger streams, from 10 to 20 ft.; and in all cases intermediate points should be taken when the conditions demand it. Mr. Hoyt.

Mr. Kuichling has raised several pertinent points in regard to the theoretical working of the meter. As the United States Geological Survey, in co-operation with the National Bureau of Standards, has just started a detailed study of current meters, in order to clear up some of the theoretical questions which are frequently being raised in regard to them, discussion on this point will be postponed until these results are available.

There has been a great deal of discussion in regard to the use of the Price meter for measuring low velocities. In river measurements, stations having low velocities are eliminated as far as possible, as a small error under these conditions makes a large percentage of error in the total results: Low-water measurements, at stations where the velocity is small at low stages, are made at points where there is a measureable velocity either above or below the regular measuring section.

In some measuring sections the velocity is small in the vicinity of the banks. In such cases the flow in the portion of the stream having a low velocity is usually but a small percentage of the total flow of the stream, so that even a large error in its determination will introduce only a small error in the total discharge. As stated in the paper, it is not recommended that the current meter be used for velocities lower than $\frac{1}{2}$ ft. per sec., and in general practice it is seldom necessary to measure velocities less than this.

It must be borne in mind that in the use of the meter the trained observer will take into account all the conditions which may affect his results, and that the meter is used as rated, so that errors due to difference of rating on a rod or cable will be eliminated.

The difference in rating between still water and moving water can only be obtained by a comparison of results of measurements made both by standard weirs and with the current meter. The data available show that such measurements check closely, and it is believed that the error due to the rating in still water is practically negligible.

In considering the accuracy of any instrument, the errors due to the observer are often confused with those due to the instrument itself. This misconception has repeatedly been made by critics of the current meter. As has been said before, the accuracy of the results obtained with this instrument will depend wholly on the personality of the man using it and on the methods used.

The work of the Water Resources Branch of the United States Geological Survey is for the development of the water resources of

Mr. Hoyt. the country, and it is principally through the engineer that the value of the data is brought out, therefore, it has been the policy of the Survey to consult freely with engineers using the results. Suggestions in regard to the work are always welcome, and every effort is being made to bring the work up to the highest standard.

In judging the results of this work, the funds which have been available for its execution should always be considered. With limited resources and a large territory to cover, it is impossible to obtain as accurate results as desired, and most of the faulty records of the Survey are due wholly to insufficient funds for obtaining better data.

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TRANSACTIONS

Paper No. 1134

THE PURIFICATION OF THE WATER SUPPLY OF STEELTON, PENNSYLVANIA.*

By James H. Fuertes, M. Am. Soc. C. E.

With Discussion by Messrs. H. V. Hinckley, William R. Copeland, James E. Little, Samuel Rideal, Gavin N. Houston, D. G. Thomas, Joseph A. Sargent, George C. Whipple, Morris Knowles, George W. Fuller, J. M. Diven, and James H. Fuertes.

The plant for the purification of the Steelton water departs, in several particulars, from previously constructed types, and embraces features of some novelty. The interest expressed by engineers and municipal authorities who have visited it, and seen it in operation, inclines the writer to believe that, although the plant is not a large one, a description of the conditions leading to its design, as well as an account of its construction and the results thus far obtained in its operation, would bring out an instructive and useful discussion of certain subjects on which more information is desirable.

The plant is a slow sand filter having a capacity of 3 000 000 gal. of filtered water daily; and among its novel features are:

1. The use of coarse-grained filters instead of subsiding basins in the preparatory treatment of the water;
2. The small area required—about one-fourth of that ordinarily required for a slow sand filter plant of equal capacity;
3. The system of handling and caring for the sand removed from the slow filters during scraping;

* Presented at the meeting of October 6th, 1909.

4. The system of determining, measuring, and applying the chemical solutions;
5. The construction of, and the filtering materials in, the roughing filters;
6. The method of operating the roughing filters, and the safeguards to prevent the deterioration of the effluent;
7. The control of the rate of filtration of the slow filters.

Among the novel features pertaining to the operation of the plant are the following:

1. The very high rate of operation of the slow filters;
2. The small quantity of coagulant necessary to secure satisfactory results—about one-fifth of that required in a mechanical filter plant of equal efficiency, and about one-eighth of that required to secure equally satisfactory results with coagulation and subsidence;
3. The ease of manipulation of the chemical dosing, and the certainty in its application;
4. The long runs of the roughing filters when no coagulant is necessary, and the consequent saving in wash-water, power, and attention;
5. The long runs of the slow filters, and the resulting large quantities of water filtered between scrapings;
6. The uniformity in the quality of the water applied to the slow filters, and consequently the satisfactory results obtained in the final purification;
7. The brilliancy of the effluent from the slow filters;
8. The absence of trouble with rusty hot water and troubles from the destruction of the solder in copper closet tanks;
9. The moderate cost of construction and operation.

The Borough of Steelton, having a population of approximately 15 000, is in Dauphin County, Pa., on the same side of the Susquehanna River as Harrisburg, and about 3 miles farther down. Topographically, the territory within the borough limits consists of a flat portion, about 20 ft. above the level of the river and from 1 000 to 1 500 ft. in width, which extends the whole length of the borough, and the residential district, which extends up the side hill to the east, and is much broken, and traversed by streets having for the

most part heavy grades. The principal street skirts the eastern edge of the flat area, the larger portion of which is occupied by the works of the Pennsylvania Steel Company.

Original Water-Works.—The original water-works plant was built by the Steelton Home Water Company, and was purchased by the borough in the latter part of 1899 for $150 069.48, since which time improvements to the intake, pumping station, and reservoir, and extensions of the street mains swelled the total cost to $223 000, on January 1st, 1906, of which $145 000 was the cost of the street mains, $38 000 the cost of the reservoir, and about $40 000 the cost of the intake and pumping station. The plant, at that time, obtained its water from the Susquehanna River through a 16-in. intake pipe reaching out approximately 1 400 ft. from the pumping station, to within about 150 ft. of the shore of Stuckers' Island. The original supply had been derived from wells dug on that island, but in a comparatively short time it became impossible to keep up the supply from the wells, and a pipe was extended about 50 ft. out into the river from the most southerly well, the water being then drawn from the river to the wells and back to the pumping station through the 16-in. pipe. Subsequently, owing to the silting up of the wells and their inaccessibility during high water in the river, a section of the 16-in. pipe was taken out at a point about 150 ft. east of the island, allowing the water to enter the pipe from the river direct; it was in this condition on January 1st, 1907.

The water flowed through this broken-into intake pipe to a brick-and-concrete-lined well situated about 150 ft. from the river, just east of the Pennsylvania Railroad tracks, and was pumped from this well to the service reservoir, 270 ft. above the river, by two 1 500 000-gal. Deane, compound, direct-acting, outside-packed, plunger pumps, through a 12-in. force main. An additional 16-in. main, connected up at the pumping station to permit either pump to pump direct or to the reservoir, parallels the 12-in. force main for about 1 200 ft., extending to Second Street on Connestoga, and is cross-connected with the distribution system and the 12-in. force main. The service reservoir, when full, holds about 7 500 000 gal.

Character of Susquehanna River Water.—The Susquehanna River, above Steelton, has a water-shed of approximately 24 300 sq. miles. It drains practically one-half of the State of Pennsylvania, and about

6 080 sq. miles in the southern central portion of New York State. Upon its banks are many large cities, including Binghamton, Cooperstown, Corning, Elmira, Hornellsville, Horseheads, and Owego, in New York State, and Athens, Altoona, Bellefonte, Clearfield, Emporium, Harrisburg, Hazleton, Huntingdon, Lewistown, Lock Haven, Mahoning, Mifflintown, Mount Carmel, Nanticoke, Pittston, Plymouth, Sayre, Scranton, Shamokin, Sunbury, Towanda, Tunkhannock, Tyrone, Wilkes-Barre, and Williamsport, in Pennsylvania. Approximately, 1 000 000 persons reside in the cities and towns along the river and its branches above Steelton, of which probably 750 000 are located along the North Branch, about 100 000 on the West Branch, and 150 000 on the Juniata, which streams unite a few miles above Steelton to form the main Susquehanna River. In addition to this urban population, there is a very large rural population, and, of the larger cities, there are upwards of 60, having a total population of about 700 000, which have sewers discharging directly into the river and its branches.

The anthracite coal fields of Pennsylvania are on the water-shed of the North Branch, and along the whole length of the river and its tributaries there are many important manufacturing industries, the wastes from which are discharged into its waters. The West Branch lies largely in a sandstone country, and the principal industries along it have been in connection with the manufacture of lumber. The Juniata lies very largely in a limestone district.

In the days of canal navigation, several dams were built on each of the main streams, canals paralleling the streams on one side, or both, between the pools formed by the dams. The principal dams on the North Branch were at Clark's Ferry, Sunbury, Nanticoke, Wilkes-Barre, and Binghamton; on the West Branch, at Lewisburg, Muncy, Williamsport, Lock Haven, and Queen's Run; on the Juniata, at Millerstown, Lewistown, Newton, Hamilton, and Huntingdon; and on the branches at Pipers, Petersburg, Big Water Street, Little Water Street, Willow Street, Donnellys, Smokers, Mud, Williamsburg, Three Mile, Crooked, Frankstown, and Hollidaysburg. The existence of these dams is mentioned because they play an important part in the modification of the character of the river water during the season of low water as well as during the early stages of the fall and spring floods. Essentially, it will be seen that these various streams consist of a series

of comparatively steep natural slopes joining level pools, which, during low and moderate stages of water, form subsiding basins in which suspended matters carried by the water may be deposited in greater or lesser amounts. During freshets, however, the deposits which may have accumulated in the pools during the preceding quiet periods are scoured loose from the bottom, picked up by the rushing waters in a more or less changed condition, and carried down the stream in large quantities.

The larger part of the water-shed is semi-mountainous, the main streams flowing, for the greater part, through unerodable and comparatively steep valleys, and the hillsides being largely denuded of forest areas; for this reason the stream and its branches are more or less flashy, big floods rising to a height of many feet in a few hours and falling almost as rapidly.

The amount and character of the sediment in a turbid water is of importance in determining the method of preliminary treatment best adapted to prepare the water for final purification. At Steelton, the quantity which will subside to the bottom of a 1-gal. bottle will sometimes reach a depth of ½ in., and is usually a composite of sand, coal dust, leaves, stones, silt, nitrogenous matter, coagulated clays, as well as aluminum hydrate and the hydrated oxides of iron mixed with various forms of dirt, bacteria, and other matters.

In the Susquehanna River water at Steelton a brown sediment, varying from red to yellow in color, either flocculent or otherwise (the flocculence depending largely on the stage of the river and the influence of the stage on the accumulation of bacterial masses and clay particles into agglomerations), is generally characteristic of the water in the west channel, although a sediment of this color may sometimes occur over the whole width of the river, following heavy general rains. A microscopic examination of this brownish sediment shows that it consists of particles of coal and dirt agglomerated by growths of small water plants and filamentous bacteria, probably of the order *Bacterium*, or *Leptothrix*.

A greenish-black sediment, resulting from the mixture of coal dust and clay, is generally flocculent, and is characteristic of the water in the east channel during heavy floods.

A grayish sediment, due either to finely-divided suspended matter other than clay, or to the hydrates of aluminum and of iron, which

settle to the bottom when the river is low, is characteristic of the water in the east channel, and is locally called "sulphur water."

A black sediment, generally more or less flocculent, consisting of finely-divided particles of coal from the mining districts, is characteristic of the east channel, and can be observed on the bottom of the river below islands and bridge piers, and in other protected positions when the river is at low stages.

All possible gradations of color intermediate between these are observable at different times owing to the commingling of the different waters.

The presence of red or yellow clay in the Susquehanna River water at Steelton, in increasing quantities, is almost uniformly accompanied by decreasing alkalinity; such water clears up slowly by subsidence. The water carrying flocculent sediment, which is generally black during floods but often the clean hydrate when the river is low, will clarify very quickly by subsidence. Such waters are characterized by low alkalinity and correspondingly high permanent hardness.

The color of the turbidity of the Susquehanna River water, therefore, is a fairly good index of its character, and, during the operation of the filters, plays an important rôle as a guide to the proper use of the coagulant.

The changes which take place in the character of the sediment, as the result of bacterial activity, are intimately associated with the iron, sulphur, and alkaline constituents of the water. Shortly after the Harrisburg experimental filtration plant was put in operation, in the fall of 1902, one of the roughing filters, then operating without a coagulant, became suddenly clogged to a condition of water-tightness within a very few minutes after the turbid raw water was turned on, and an examination disclosed a heavy blanket of black, sticky mud on the surface of the sand. It was surmised that this mud might be similar to that which was reported to have accumulated in the sedimentation basins at Cincinnati during the experiments conducted by George W. Fuller, M. Am. Soc. C. E., but a chemical examination of the mud on the Harrisburg filter showed that while iron was present in considerable quantities it was not in the sulphide form. The mud was found to consist essentially of very fine particles of coal dust mixed with the hydrates of iron and aluminum.

Although it had been understood for a long time that the drainage

from coal mines contained a considerable quantity of free sulphuric acid, an explanation of the processes by which this was formed was not generally current. Recognizing that ferrous sulphide, as pyrites, was present in the coal dust in the river, it was suspected that the "sulphur bacteria" and "iron bacteria," which work various transformations in iron compounds, might play some part in the formation of this sulphuric acid, and an experiment was made to test the hypothesis. A small quantity of coal dust or culm was pulverized in a mortar, and equal portions were placed in narrow-mouthed bottles. In each of these bottles was then placed a definite quantity of untreated river water, and, the mouths of the bottles having been closed with cotton plugs, half the number were sterilized in an autoclave at a pressure of 15 lb. for 30 min. After standing in the laboratory for about a month the bottles which had not been sterilized exhibited very different characteristics from those which had been so treated. After both sets of bottles had been shaken violently the turbidity in the set which had not been sterilized was observed to decrease very much more rapidly than that in the set which had been sterilized, somewhat after the manner that the untreated river water containing this hydrated oxide of iron acts in comparison with water which does not contain it. The water in these bottles represented, in some degree, the condition of the water in the river, except that the bottles which had been sterilized contained no living bacteria, while the others, supposedly, did, having been filled with river water which at the time of filling was known to contain a large number. The results of this preliminary experiment seemed to indicate that the change which had taken place in the unsterilized bottles was due to the action of the bacteria on the iron compounds contained in the culm.

Following out this idea, a further elaborate set of tests finally led to the conviction that a hydrate of iron was formed from the pyrites, in the presence of bacteria, while it was not formed when bacteria were absent; further, there was no evidence of the formation of soluble iron except when bacteria were present.

The reactions which took place in the bottles used in the experiments are not known, and are probably of a very complicated character, some resulting directly from, or being intimately associated with, the biological activities present, and others in all probability being spontaneous chemical reactions. It would be foreign to the subject of

this paper to enter into a discussion of these experiments in detail and of others which have since been made, both on Susquehanna and on Schuylkill waters; it will be sufficient to state that at the conclusion of the Harrisburg experiments the data indicated that the bacteria occurring normally in the Susquehanna River, in the presence of organic matter on which they can live, and under proper conditions of storage, are able to transform the pyrites in the coal into compounds which furnish the material for the hydrated oxide of iron naturally occurring in the water. The indications seem to be that the sulphur in the pyrites, in contact with water, furnishes the material from which the bacteria make sulphuric acid, which, coming in contact with the iron, forms the soluble ferrous sulphate. The water containing this soluble ferrous sulphate commingles with the alkaline waters of the river farther down stream, the sulphate of iron being decomposed, the sulphur combining with the lime and magnesia to form the sulphates of these bases, and the iron being converted into the hydrate and subsequently into the hydrated oxide. This seems to be at least one method by which free sulphuric acid is formed in the drainage of the mining regions; the process also takes place to a very considerable extent in the level reaches of the river, where culm is deposited as floods recede. When the river rises, and the dirt on these flats is washed away, there is borne with it the hydrate and hydrated oxide of iron which had been formed during exposure. At the Clark's Ferry dam, some miles above Harrisburg, the water in the east channel sometimes shows this coagulated condition so plainly that frequently within an area of a few square feet one may collect a sample of either perfectly clear, or very black water.

At times of very low water there occurs along the east shore a brownish flocculent sediment composed of the hydrate of aluminum as well as the hydrate of iron. The source of the aluminum entering into the composition of this hydrate is unknown; possibly it is derived in some way from the clays.

As a result of the natural formation of the hydrated oxide of iron, and of aluminum hydrate, in the water of the river, a considerable degree of apparent self-purification takes place during the summer months. This is brought about, in the pools above the dams, by the entangling of the bacteria into flocculent masses and their consequent settlement to the bottom of the river with the other suspended

matter. The action is, on a large scale, very similar to the results of the treatment of sewage by chemical precipitation, and the active precipitants are the same as those used in the artificial process. Ultimately, a large proportion of the organisms thus carried to the bottom cease to exist in an active condition, finding their environment not suited to sustain their vitality. The apparent purification is at times so great that, although from the many cities along its course enough sewage is flowing continuously into the river to render the water at its extreme low stage offensive, yet at such times the water, as it flows past Steelton, is actually at its best, both in appearance and as to its hygienic qualities. The analyses of different samples of river water collected at Harrisburg during low water have shown as few as 10 colonies of bacteria per cu. cm., with no intestinal bacteria present; and generally, during low-water stages, when the quantity of sewage discharged into the river is greatest relatively to the stream flow, the numbers of bacteria in the river water opposite Steelton (from points far enough away to be beyond the influence of the Harrisburg sewage) are lowest and the numbers of intestinal bacteria fewest. This condition obtains, however, only during dry weather, for the first floods which come down in the fall scour out a portion of the deposits from the pools formed by the dams up stream, and then the counts show sometimes as many as 300 000 colonies of bacteria per cu. cm. of water, as well as large numbers of bacteria characteristic of intestinal discharges.

The total hardness of the river water varies with the stage of the river, yet not necessarily in proportion thereto, and is about the same, as a general thing, from bank to bank. The water is hardest at low stages when the flow is largely from springs, that on the west side being high in carbonates and low in sulphates while that on the east side is low in carbonates and high in sulphates, owing to the chemical changes taking place, as above described.

These modifying conditions destroy any relationship between turbidities, river stages, and numbers of bacteria. High bacterial counts sometimes accompany very low, as well as very high, turbidities, and, similarly, relatively low turbidities are sometimes found at very high stages of the river; further, the color of the turbidity is no indication of its total amount, as a comparison of the records of ten samples of water, each with a turbidity of 1 000 parts per million, collected at

different times, and at different parts of the river, shows all gradations of colors between bright yellow, red, dark brown, and black.

As a result of these complex conditions, the water in the river may change quickly from a moderately hard, clear water to a soft water carrying several thousand parts per million of turbidity, which, on occasions, may intermittently come down coagulated to a more or less slimy or flocculent condition with hydrated oxide of iron and hydrate of alumina. On the other hand, the river from bank to bank may be carrying its maximum load of yellow clay from the Conodoguinet and Juniata, or reddish-brown mud from the West Branch. During a general flood from the entire water-shed, all these conditions may prevail at the same time, the yellow water from the Juniata keeping on the western side, the West Branch water keeping in the central part, and the black water from the North Branch along the eastern shore, each stream occupying practically one-third the width of the river, which, opposite Steelton, is approximately four-fifths of a mile.

Although of great width, the river is comparatively shallow, the bed in that vicinity being of shale and limestone, and the depth of the water at its lowest stage being not more than 3 ft. at any point and not more than 1½ ft. for the greater part of the width. Some trouble is caused during the winter by slush-ice and anchor-ice, the formation of which is favored by these conditions.

The Steelton Filter Plant.

Those who have had experience in the filtration of polluted waters will appreciate some of the difficulties encountered in the purification of the Susquehanna River water, particularly as to bacterial purification, for the reason that the numbers of bacteria in the raw water may be increasing rapidly while the turbidity is decreasing; and hence, if the attempt is made to control the bacterial purification by operating the plant on the indications of the removal of turbidity only, the result may be anything but satisfactory.

Another complication arises from the exhaustion of the alkalinity of the raw water by the acid mine wastes, and also its reduction by dilution during heavy freshets. This, however, can readily be taken care of by adding enough lime or soda-ash to supply the deficiency in alkalinity, in case a coagulant is used in connection with the process

of purification. Among other complications is the difficulty of securing the satisfactory coagulation of the raw water when the bacterial counts are high and the turbidity is low, and also the difficulties encountered in securing a sufficiently heavy coagulum when the temperature of the water approaches the freezing point, at which times the turbidities are also usually low.

While all these difficulties can be overcome and the water can be handled successfully with a mechanical filter plant such as that installed for the City of Harrisburg, the successful operation of such a plant depends on the maintenance of a fully equipped laboratory, and on a superior grade of supervision; and, to secure these, it is essential, of course, that the revenues derived from the sale of the filtered water be sufficient to cover the additional expense.

Owing to the considerable general pollution of the river, the discharge of the sewage of Harrisburg into it only a couple of miles above Steelton, the excessive turbidity, and the enormous quantities of sand and coal delivered into the borough's distributing reservoir by the supply pumps (ultimately to enter the distribution mains, interfering with the action of meters, and clogging service pipes), the question of securing a purer and more satisfactory source of supply was brought to an issue in the winter of 1906, and the writer was commissioned to investigate and report on what could be done toward securing a new supply of a satisfactory quality, or toward purifying the existing supply. The report recommending the purification of the present supply was submitted to Councils early in 1907, and, following its adoption, the borough voted to issue $80 000 in bonds to cover the cost of the necessary works.

The plant, as built, was designed to place the securing of satisfactory results, as to purification, within the reach of the Water Department without the necessity of maintaining a laboratory, and without incurring unduly heavy construction and operation costs. The general basis of the design is as follows:

First.—The removal by subsidence of such of the suspended matters as will settle out from the raw water in about 12 min.;

Second.—The removal, by passing the water rapidly through deep, coarse-grained filters, of at least 90% of the applied turbidity, using a coagulant to secure this result when necessary, and allowing no coagulated turbidity greater than 25 parts per million to issue from

the coarse-grained filters, no matter what the turbidity of the applied water;

Third.—The filtration of the effluent of the coarse-grained filters through slow sand filters at a relatively high rate.

New Intake.

The works include a new 30-in. cast-iron intake pipe extending out about 1 500 ft. into the river to a point where observations indicated that there would be a minimum of trouble with silt, sand, and coal dust. At the point where the intake was located prior to the commencement of these improvements large quantities of sand and fine particles of coal were drawn into the intake well during floods in the river, as much as a car load a week requiring removal in order to keep the pumps in operation.

The new intake pipe was laid to proper line and grade, in cofferdams, in a trench excavated in the rock bottom of the river, the joints being poured with lead and caulked water-tight.

The intake cage consists of a reinforced concrete structure inclosing the end of the intake pipe and spreading out like a fan to give an opening in front 1 ft. high and 20 ft. long, with vertical screen bars of 1-in. round steel, spaced 6 in. apart, in two rows, and staggered. The top of the opening was located at a depth of 1 ft. below extreme low water, and the rock bottom of the river was blasted away in front of the cage, a smooth concrete apron being laid thereon to assist the current in rolling sand and coal particles along past the intake. The line of the face of the intake is parallel with the trend of the current of the river at that point.

A valve is placed in the intake pipe, at the shore end, and a connection is made with the existing 24-in. pipe leading to the suction well. The work on the intake, which was commenced on July 1st and completed early in September, 1908, was done by the Water Board by day labor, from the plans and under the general supervision of the writer.

Improvements at the Pumping Station.

New Pumps.—The plans, as approved by Councils on July 10th, 1907, called for the construction of a 3 000 000-gal. slow sand filter plant with roughing filters, to be built on a piece of ground 100 ft. wide and 265 ft. long, situated about 700 ft. east and 900 ft. north of

the pumping station. Before the construction of the works had been authorized, appropriations had been made for the installation of a new 3 000 000-gal., Heissler, compound-condensing, crank-and-fly-wheel, pumping engine at the pumping station.

In order to suit the new conditions, it was necessary to remodel the piping in the pumping station and install two 12-in. centrifugal pumps, direct-connected to simple, vertical, condensing engines of the marine type, the suction pipes of the new pumps being connected to the remodeled original suction to which the suctions of the old Deane pumps were attached. The discharge pipe from the two centrifugal pumps was piped to the filter plant, with provision for a cross-connection to the suction pipe of the new Heissler pump, so that, in case of accident to the new filtered-water well, raw water could be pumped directly into the Heissler pump and thence to the city. The Deane pumps were located in a pit sufficiently low to enable them originally to take their own suction from the intake well, but the new Heissler pump, as well as the remodeled Deane pumps now take the filtered water from a new filtered-water well constructed just outside of the pumping station.

From this brief description it will be seen that in the remodeled plant the raw water is pumped from the intake well by centrifugal pumps to the filter plant, and the filtered water is returned to the new pump well just outside of the pumping station, from which either the new Heissler or the old Deane pumps can draw the filtered water and force it through a 12-in. force main to the distribution reservoir on the hill above the town, or through the 16-in. return main directly into the distribution system, or into the reservoir and the distribution system at the same time by properly manipulating the control valves on the cross-connections. By this arrangement the construction of an additional force main to the reservoir, which would have been an immediate necessity owing to the increased consumption of water, was deferred for many years by reason of the provision of a pure water which could be pumped into the mains direct.

Raw-Water Delivery and Filtered-Water Return Mains.—The raw-water pipe leading from the pumping station to the filter plant, and the return pipe leading from the filter plant to the new pump-well are of wood-stave construction, with cast-iron specials for bends and connections to the concrete.

The staves were of Oregon pine dressed to the true circle and constructed in the usual manner with steel tongues in the end joints of abutting staves, and ½-in. round steel bands, with rolled threads and standard hexagonal nuts engaging against cast malleable-iron shoes, the bands being placed 12 in. apart, excepting at butt-joints and on the ends of cast-iron specials, where the spacing was reduced to about 4 in. The shoes and bands were dipped in a hot bath of Pioneer mineral-rubber pipe coating, and were retouched, in the trench, after cinching, with Smith's Durable Metal Coating applied with a brush.

The raw-water and filtered-water pipes have an internal diameter of 24 in., and, for the greater part of their length, are laid in one trench.

Filtered-Water Well.—The new pump well at the pumping station is of concrete. It is circular in plan, has an internal diameter of 15 ft., and a depth of 14 ft. from the floor to the underside of the reinforced roof. It has a cast-iron manhole frame and cover, the opening being 36 in. in diameter; the cover is provided with a gasket and is bolted down water-tight to the frame, which is built into the concrete.

Where the 14-in. discharge pipe from the centrifugal pumps passes through this well to form the connection with the wood-stave pipe, a by-pass connection is made by which the raw water can be discharged into the filtered-water well in case of necessity, but the valves and pipes are arranged so that any leakage in or through the valve on the raw-water pipe will drain out to the raw-water well and not contaminate the filtered water.

Purification Works.

Capacity of Plant.—The purification works include a deposit chamber and three roughing filters which, with their regulating wells and the regulating wells for the slow filters, occupy a space approximately 65 ft. long and 44 ft. wide; in addition there are three slow filters which occupy the remaining available space, 100 ft. wide and 200 ft. long.

The plant is designed to supply 3 000 000 gal. of filtered water per day, a proper reserve capacity being provided to permit of cleaning the filters while delivering the stated quantity of water.

Method of Purification.—The method of purification consists in

PLATE XIII.
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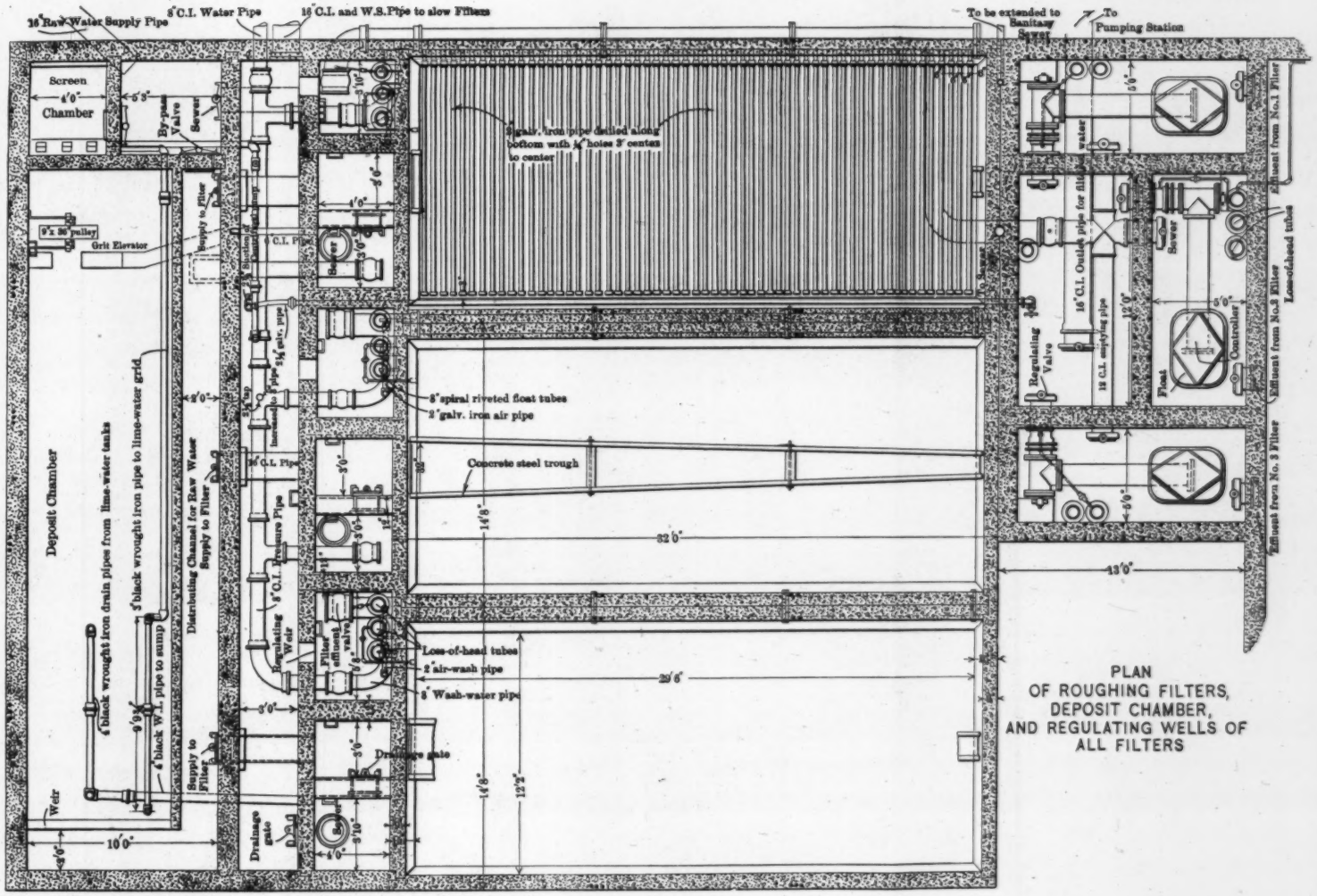

PLAN
OF ROUGHING FILTERS,
DEPOSIT CHAMBER,
AND REGULATING WELLS OF
ALL FILTERS

preparing the raw water for slow filtration by removing therefrom enough of the turbidity and bacteria to permit the slow filters to be operated at a relatively high rate. The object of the treatment is to prepare the water in such a way as to be able to discharge upon the slow filters, for final purification, a water of reasonably constant composition, as to turbidity and bacterial contents, throughout the entire year, keeping the maximum limits of turbidity and numbers of bacteria in the applied water down below figures which experience indicates to be desirable.

The treatment begins with the removal of the floating matters from the water by coarse screening and of the particles of sand, silt, and coal dust by subsidence. The water is then passed through roughing filters containing a deep bed of coarse-grained filtering material in which an additional portion of the remaining suspended matter (including a varying percentage of bacteria) is retained, the effluent water passing to the slow sand filters for final purification. As a general rule, when the numbers of bacteria in the raw water are relatively low and when the turbidity does not exceed 50 parts per million, the roughing filters produce an effluent of satisfactory quality without the use of artificial coagulation; but when the turbidity of the raw water is higher than 50 parts per million, or when it is caused by particularly fine particles, or when the bacteria number more than about 5 000 per cu. cm. in the raw water, a sufficient quantity of coagulant is mixed with the incoming raw water, as it enters the deposit chamber, to produce a rough filtered effluent with a turbidity of not more than 10% of that of the applied water, and not more than 25 parts per million, in any case. Adherence to these limits, which are as yet only tentative, will apparently give sufficiently long runs of the roughing filters to permit of easily keeping the plant in operation, and sufficiently long periods of operation of the slow filters to enable them to pass a satisfactory quantity of filtered water between scrapings.

The period of subsidence allowed in the deposit chamber, for the maximum capacity for which the plant is designed, is 12 min. The roughing filters are designed to operate at a rate of 172 000 000 gal. per acre of filter surface per day when delivering 1 500 000 gal. per filter, three filters being provided so that two can yield the full 3 000 000 gal. while the third is being washed. The net area of the sand surface of each slow filter is 0.1446 acre, and when delivering 1 500 000 gal.

per day the filter will operate at a rate of 10 373 000 gal. per acre per day, two filters yielding 3 000 000 gal. daily with the third in reserve for cleaning.

Deposit Chamber.—The raw water is received at the filter plant at the bottom of a rectangular compartment in one end of the reinforced concrete deposit chamber, Plate XIII, a valve-controlled connection being provided between the bottom of this compartment and the sewer. One outlet from the upper part of this compartment leads to the deposit chamber and a second leads directly to the channel feeding the water to the roughing filters. By using one or the other of these outlets the water may be passed through the deposit chamber, or, if desired, the deposit chamber may be by-passed and the raw water delivered directly to the roughing filters. After leaving the inlet chamber, on its way to the deposit chamber, the water first passes vertically downward through a coarse screen composed of parallel lines of 2-in. planks standing on edge, horizontally, and spaced about 8 in. apart from center to center transversely to the direction of the entering water; between these planks, and in a vertical plane, one above the other, are the horizontal perforated pipes through which the lime-water and coagulant solutions are admitted to the raw water.

The raw water is admitted to the deposit chamber in the manner described, in order to use up the entering velocity head and permit the water to pass through the deposit chamber quietly and thus drop out as much as possible of the suspended matter in the short time allowed therefor.

Grit Elevator.—Near the end at which the raw water enters, a grit elevator is installed to remove continuously the sand and coal particles which may collect in the deposit chamber. The elevator, Plates XIII and XIV, which has an estimated capacity of 20 tons per 24 hours, consists of an 8-in., 6-ply rubber belt carrying perforated, galvanized-iron buckets, 6 in. long, 4 in. wide and 4 in. deep, at intervals of 2 ft. The belt travels at the rate of 25 ft. per min., and the buckets discharge into a hopper arranged to fit into the end of a 6-in. cast-iron pipe leading to one of the sewer compartments of a roughing filter.

The deposit chamber is 10 ft. wide and 42 ft. long, including the entrance chamber and screen box, and at full capacity the water flows through it with a depth of about 9.2 ft. and a velocity of 0.05 ft.

PLATE XIV.
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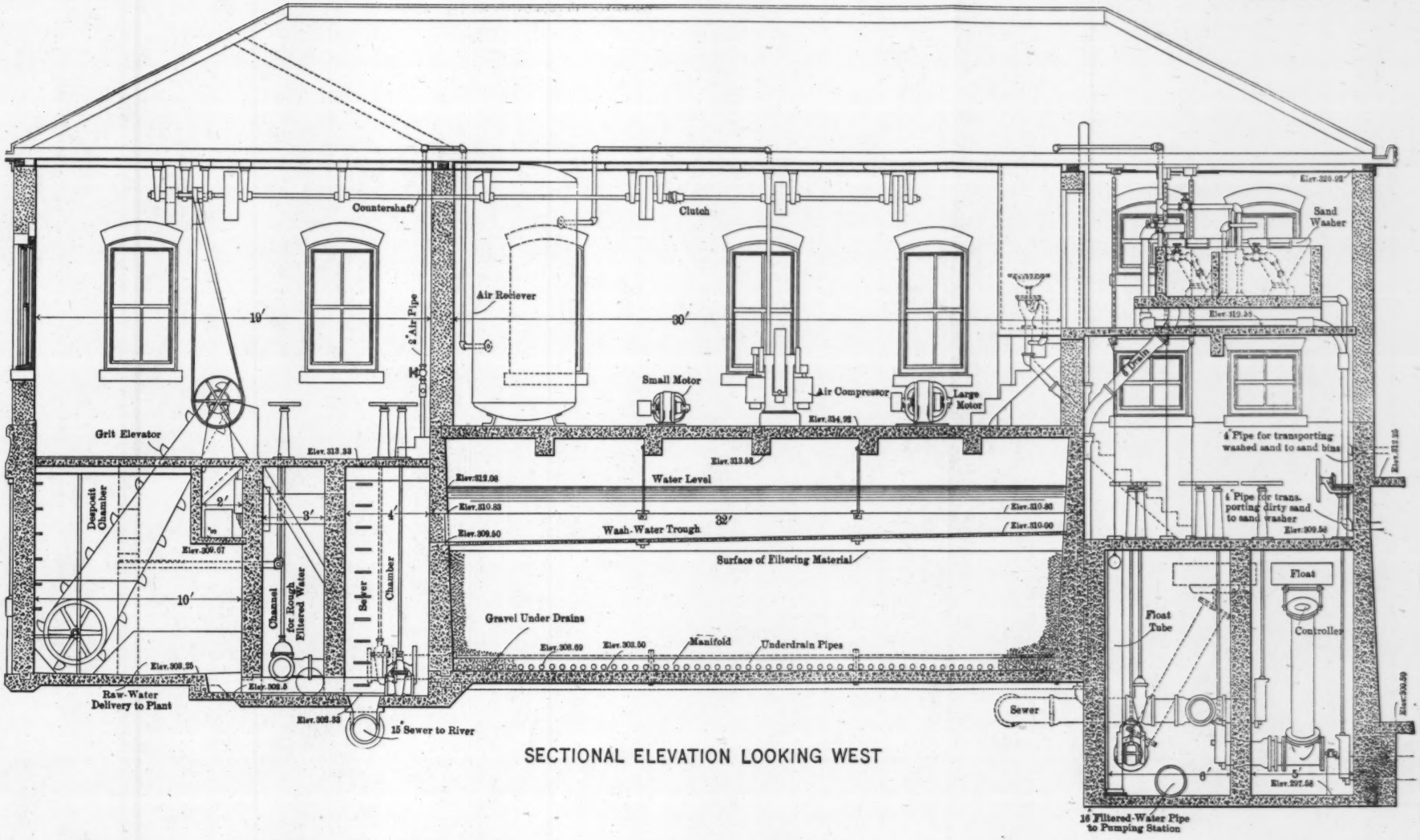


SECTIONAL ELEVATION LOOKING WEST

per sec., passing over a weir at the farther end and into a channel leading to the inlet openings to the roughing filters.

Chemical-Mixing Tanks.—Vertically above the deposit chamber, and carried on heavy reinforced concrete beams spanning the chamber, stand the mixing tanks, Plates XV and XVI, for the lime-water and coagulant solutions. These tanks are of monolithic reinforced concrete. In each tank there is a wooden dissolving rack with an open bottom carrying wooden frames provided with copper screens having 20 meshes per inch, reinforced by a coarse copper screen having 4 meshes per inch, with No. 11 bars. Each tank is provided with a system of $\frac{3}{8}$-in. perforated copper pipes at the bottom connecting with a line of galvanized-iron pipe leading from above the tops of the tanks to the air receiver, to provide for the agitation of the solution after it has been mixed. A galvanized-iron steam connection, with a copper branch reaching to the bottom of the coagulant tanks, is provided for warming the solution during cold weather. The solution is mixed in the usual manner. With a 2% solution, two tankfuls will furnish a dose of 2 gr. per gal. for a consumption of 3 000 000 gal. daily. The coagulant solution is drawn from the mixing tanks through 2-in. copper pipes arranged with swivel-joints to permit of decanting the solution at any desired depth.

Coagulant-Measuring Box.—Through the copper outlet pipes the solution runs to the coagulant-measuring box, Fig. 1, and Fig. 1, Plate XVII. This is a reinforced concrete box with a plate-glass front in which are drilled three $\frac{1}{4}$-in. and three $\frac{1}{2}$-in. orifices, all at the same elevation and spaced 4 diameters apart and 4 diameters of the $\frac{1}{2}$-in. holes from the bottom and side edges of the plate. This glass plate is 11 in. wide, $\frac{1}{4}$ in. thick, and 28 in. high, and the holes are countersunk one-half the depth of the plate from the outside. On the back of the plate is etched a graduated scale reading to hundredths of a foot for a height of 2 ft. above the centers of the orifices. The plate is fastened to the front of the orifice box on soft rubber gaskets, made of tubing, by brass plates and expansion bolts secured in the concrete. The depth of the solution in the orifice box is regulated with a bronze lever valve and float arranged to admit the solution near the bottom and permit the regulation of the depth in the box by changing the height of the float. The float is of copper, 10 in. in diameter, with a flat bottom and a coned top, sliding on a $\frac{1}{4}$-in. brass rod, the rod being

hinged to the lever of the valve, and the height of the float being adjusted on the rod with a bronze set-screw having a milled head.

Each orifice is provided with a soft rubber cork, or stopper, attached to a handle consisting of a piece of brass pipe with a cap on one end and an elbow on the other, the cork being fastened to a nipple entering the elbow; there is a rack, with thumb-screws, across the top of the orifice box, for holding the handles where desired.

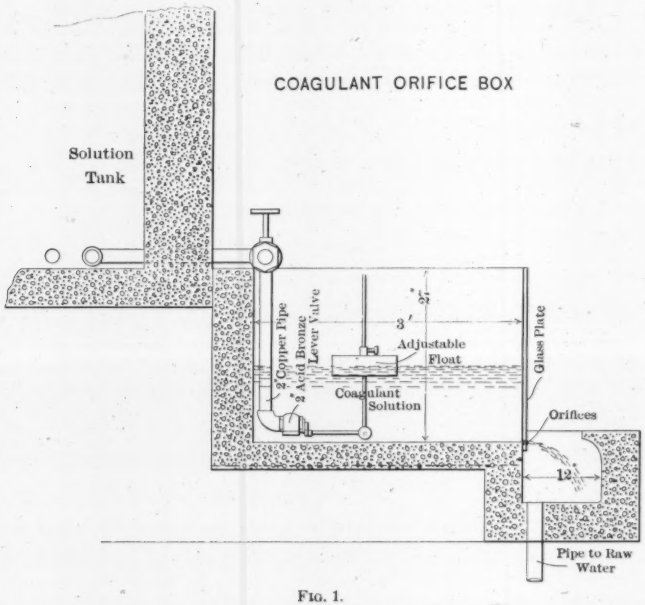
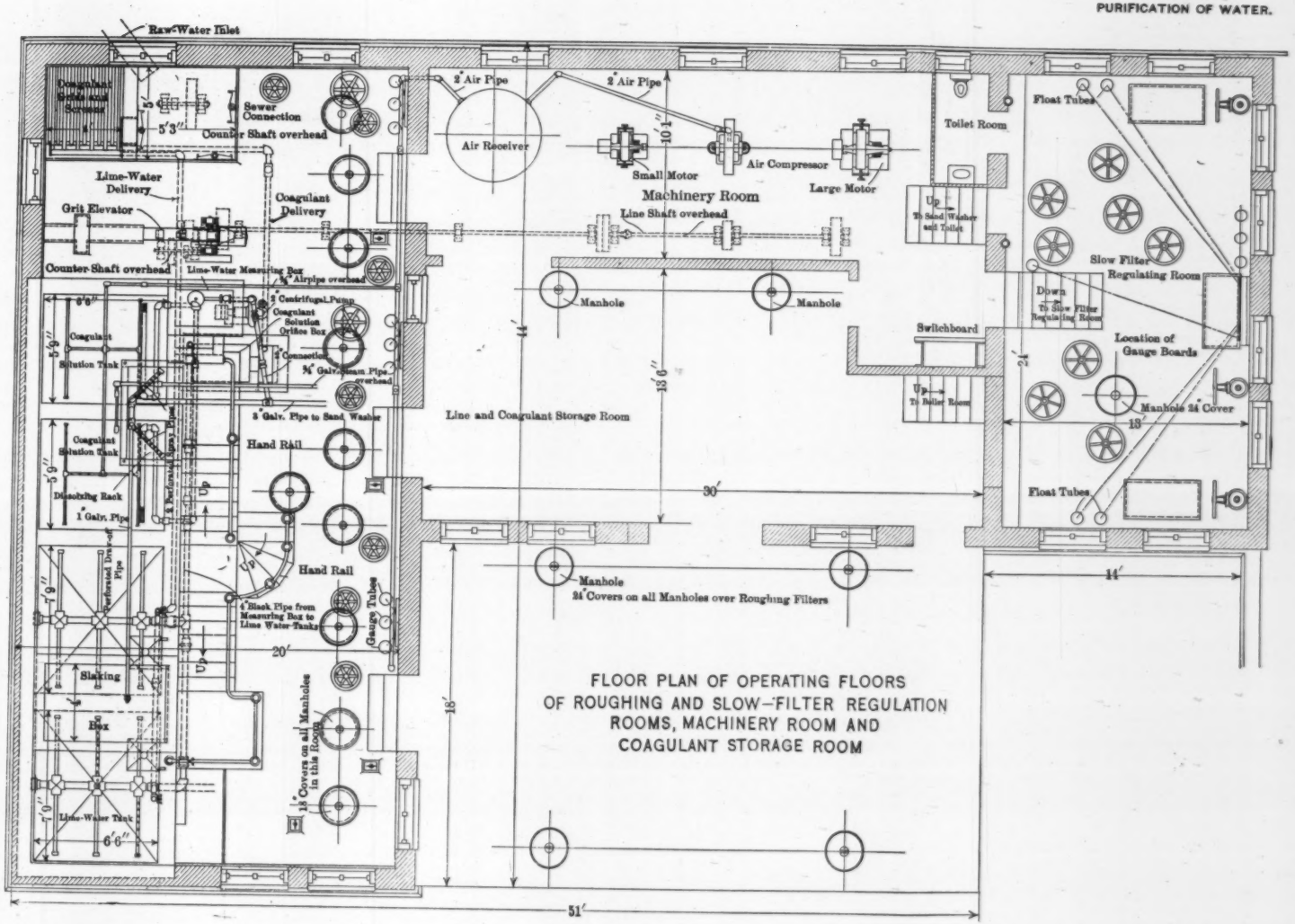


FIG. 1.

The coagulant solution flows through the orifices in the glass plate to a small concrete box, from which a 2-in. copper pipe connects with the perforated pipes between the screen bars at the entrance to the deposit chamber.

Lime-Water Mixing Tanks.—The lime-water apparatus consists of two tanks for making the lime-water (Plates XV and XVI), a slacking box, and an orifice box for measuring the quantity of water to be supplied to the lime-water tanks. Lime being only slightly soluble in water, a relatively large quantity of solution is required



FLOOR PLAN OF OPERATING FLOORS OF ROUGHING AND SLOW-FILTER REGULATION ROOMS, MACHINERY ROOM AND COAGULANT STORAGE ROOM

unless the lime be added as cream of lime. In the Steelton plant the apparatus is designed to make use of a saturated solution of lime-water.

The lime is slaked in a reinforced concrete box supported on top of the partition separating the two lime-mixing tanks, a sluice-gate being provided on each side for emptying the milk of lime through funnels into pipes leading down to the bottom of the lime-mixing tanks. The latter are rectangular in plan, the bottom of each being in the form of an inverted frustum of a pyramid. From the bottom of each a pipe leads to the sewer, with a connection immediately below the bottom of each tank leading to the lime-water measuring box which stands on brackets above the coagulant-mixing tanks and receives its water through a float and lever valve (with spindle and float as described for the coagulant orifice box) either from the city mains or from a 2-in. centrifugal pump taking its suction from the channel containing the rough-filtered water. Valves in the various pipes permit of cleaning the tanks and pipes, and the admission of water to either of the tanks as desired. The lime-water orifice box is also of reinforced concrete, with a plate-glass front, $\frac{1}{2}$ in. thick, 20 in. wide and 28 in. long, containing two $\frac{1}{2}$-in., two 1-in., and one $1\frac{1}{2}$-in., orifices; there is also a graduated scale etched on the back of the glass, the details of the arrangements being similar to those for the coagulant orifice box. The effluent from the solution-measuring box issues vertically into the mixing tanks through the center of the bottom, rising as a saturated solution through the milk of lime, which has been mixed and deposited in the bottom of the tank. The quantity of water rising through the tank is varied from time to time in accordance with the different conditions of the raw water and the varying consumption of water by the borough, the maximum rate, for a dose of 2 gr. per gal., for a consumption of 3 000 000 gal. daily with both tanks in service, being such that the water would rise through the tanks at the rate of 7 in. per hour if the outlet valves were closed. The water, having picked up its lime at the bottom of the tank, is skimmed off just below the surface by a system of perforated, horizontal, galvanized-iron pipes.

Application of Chemicals.—For mixing the lime-water with the raw water, six lines of $1\frac{1}{2}$-in. galvanized-iron pipes, perforated with double rows of $\frac{5}{16}$-in. holes $3\frac{1}{2}$ in. apart, screwed into a 4-in. galvanized-iron manifold connected with a loose-sleeve joint to the lime-water

supply pipe, lie between the wooden bars of the screen at the entrance to the deposit chamber. The copper pipes for distributing the coagulant solution consist similarly of six lines of 1-in. pipes perforated with double rows of $\frac{3}{16}$-in. holes spaced $3\frac{3}{8}$ in. apart and fastened to a $2\frac{1}{2}$-in. copper manifold; they lie vertically below the lime-water mixing pipes.

Roughing Filters.

The raw water, after passing through the deposit chamber, or in case it is by-passed around the deposit chamber, enters a channel, running lengthwise along one side of the deposit chamber from the

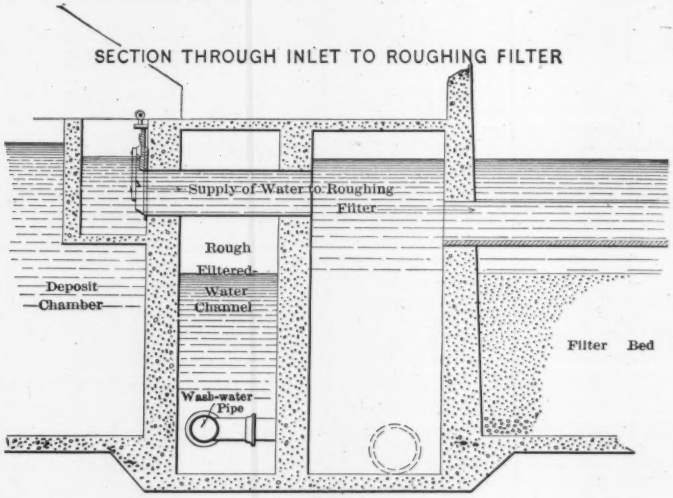


Fig. 2.

weir at the east end to the entrance chamber at the west end, the side and bottom of this channel being supported by vertical steel bars built in the concrete and passing continuously through the side of the channel and the beams spanning the deposit chamber and carrying the mixing tanks. From this channel, Plate XIII, the water is taken to each of the three roughing filters through 16-in. cast-iron pipes controlled by 16-in. sluice-gates operated with chain-hoists. Between the ends of the three filters and the east side of the deposit chamber are arranged a series of chambers, three for each filter, containing connections to the sewer, to the filtered-water effluent pipe, and to the raw-water supply, a connection being made at the bottom, between this

PLATE XVI.
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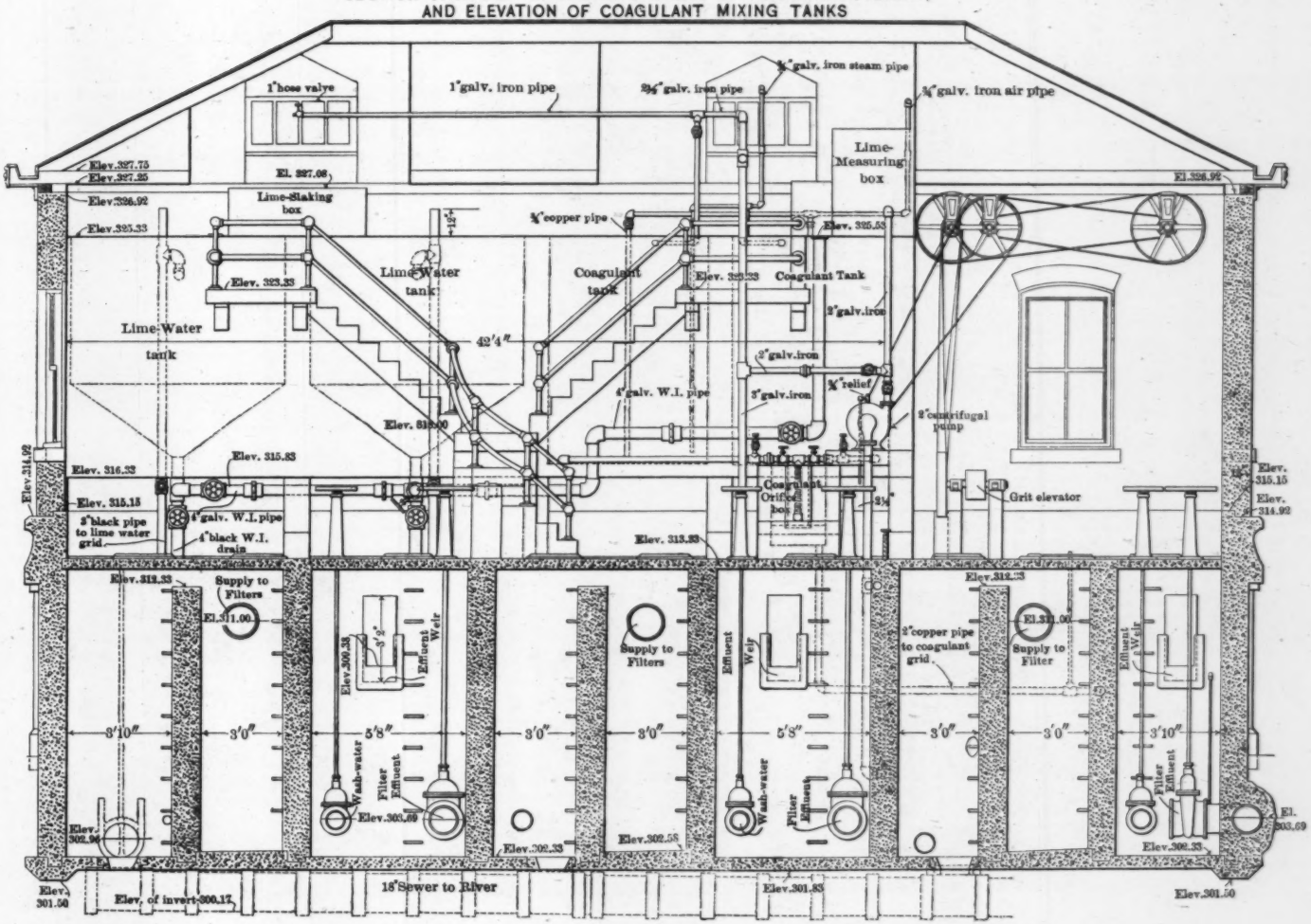
SECTION THROUGH REGULATING WELLS OF ROUGHING FILTERS, AND ELEVATION OF COAGULANT MIXING TANKS



latter chamber and the chamber leading to the sewer, the purpose of which is to permit the dirty water resulting from the washing of the filters to flow backward through this inlet chamber to the sewer leading to the river. Between the row of regulating chambers for the filters and the deposit chamber is a narrow channel, 3 ft. wide, and about 10.5 ft. deep, into which the rough-filtered water is discharged from the roughing filters, and from which the supply pipe leads to the slow filters.

The 16-in. supply pipes to the roughing filters cross the channel containing the rough-filtered water, and enter the supply chambers immediately in front of the center of each roughing filter. The wash-water troughs, which serve the purpose of distributing the incoming water over the filters during operation, as well as of removing the wash-water when the filters are being cleaned, are suspended from the roof beams of the roughing filters by 1-in. bolts engaging saddle pieces of flat steel, over the top and under the bottom of the troughs, with jamb-nuts at top and bottom to permit of adjustment.

The roughing filters are each 12 ft. 2 in. wide and 29 ft. 6 in. long at the bottom, the sides and ends battering outward 6 in. in the height of the filter, the outside surfaces of the walls being vertical. The side walls and roofs of the filters are of reinforced concrete, and the bottoms of concrete without reinforcement.

Roughing-Filter Underdrains.—In each filter there is an underdrainage system, similar in principle to that designed by the writer in 1902 for the Harrisburg Filter Plant. It consists of 59 parallel 2-in. galvanized-iron pipes, extending across the width of the filter and drilled along the bottom with ¼-in. holes 3 in from center to center, the pipes entering a manifold made of 12-in. flanged pipe built into the concrete wall on one side of the filter and terminating in the effluent regulating chamber in front of the filter in a 12-in. gate-valve operated by hand from the floor above. The 2-in. galvanized-iron pipes are placed in the filter so that their bottoms stand 1 in. above the floor of the filter, and are surrounded with a 4-in. layer of gravel composed of stones which will pass through a screen having ¾-in. meshes in the clear between the wires and remain on a screen having ¼-in. meshes in the clear between the wires, and containing no particles finer than ¼ in. in largest dimensions. Upon this rests a second layer of fine gravel, 3 in. thick, all the particles of which will pass through a screen having ¼-in. meshes, but will remain on a standard brass sieve having

12 meshes per lin. in., and containing not more than 2% of particles which will pass through a standard sieve having 15 meshes per lin. in. These specifications were written for local materials, and might require modification for other materials of different subsiding values.

Filtering Materials in Roughing Filters.—The filtering material in the roughing filters consists of a layer of fine anthracite coal screenings, 5 ft. thick, prepared from particles of fine coal washed down the river from the culm piles of the mines on the North Branch water-shed during floods and recovered from the river in the neighborhood of Harrisburg and Steelton by centrifugal pumps. Considerable care was required in the selection of the raw material, as it was desired that the finished product should have an effective size of about 1 mm. and a uniformity coefficient as low as practicable without making the cost of preparation too great. The material as received at the plant contained considerable moisture in the pile and was prepared by casting over a screen having 4 meshes per in., the actual separation with the wet coal being practically the same as would be obtained by screening the dry material through a sieve having 6 meshes per in. Each of the three filters contains 5 ft. in depth of this screened coal, which, as prepared, has an effective size of 1 mm., a uniformity coefficient of about 2.4, and does not contain more than 2% of particles which would pass through a screen having 30 meshes per in. In preparing the river coal, 144 tons of the finished product, weighing about 53 lb. per cu. ft., was required, to secure which it was necessary to screen 176 tons, rejecting 32 tons of coarse particles.

Roughing-Filter Controller.—The rate of filtration of the roughing filters is controlled by regulating the depth at which the rough-filtered water flows over the edge of standard bronze weir-plates, 12 in. long, Plate L and Fig. 3, the weirs being placed in orifices in the front walls of the effluent regulating chambers of the filters, the rough-filtered water falling over the weirs and into its channel, from which it is conducted to the slow filters, the elevation of the weirs being fixed so as to allow losses of head up to about 2½ ft.

Loss-of-Head and Rate Gauges.—The rate of filtration and loss of head are indicated, for each filter, by gauges and indexes operated by spherical, seamless, spun-copper floats, resting on the water in three 8-in. spiral riveted tubes, Plate XIII and Fig. 3, placed in the regulating chamber of each roughing filter.

FIG. 1.—ROUGHING-FILTER OPERATING ROOM.

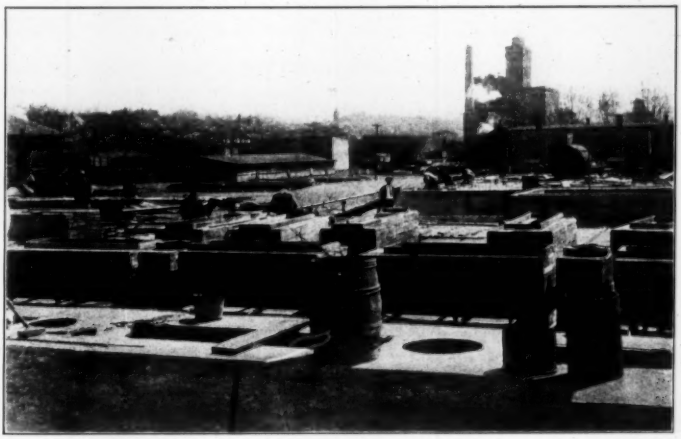


FIG. 2.—FORMS FOR ROOFS OF SLOW FILTER.

The rate gauge was graduated at the plant from the actual measured discharge over each weir in stated periods of time, and at different depths of flow, as indicated by the float gauge. The loss-of-head gauge was divided into feet and hundredths.

The gauge boards were made of strips of poplar, painted with three coats of white-zinc paint, graduated by hand with indelible India ink, the figures and letters being lead pattern-makers' letters attached to the gauge boards with shellac, on the first coat of paint, and painted white with the board before the latter was graduated. After graduation the tops of the letters were blackened, and the board was given two coats of spar varnish and stiffened by small brass cleats

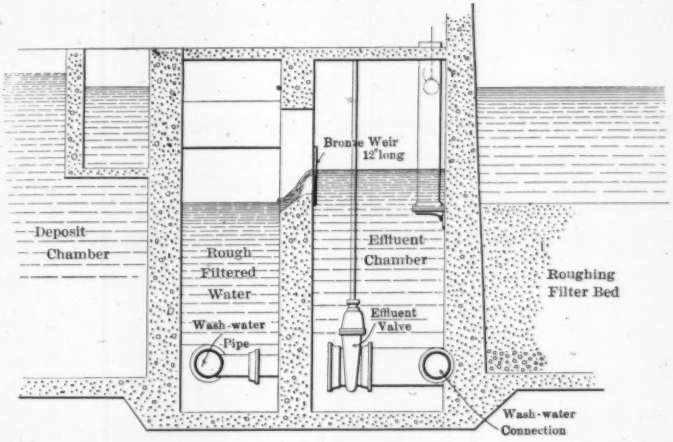


FIG. 3.

screwed to the backs at the top and bottom, the cleats being provided with ears on each side drilled with holes, $\frac{1}{24}$ in. in diameter, parallel to the back of the board and lengthwise of the same. The gauges were strung on parallel guides formed of No. 20, B. & S. gauge, phosphor-bronze wire fastened to screws at the top and bottom in the head and foot blocks.

Roughing-Filter Washing Devices.—Provision for washing each roughing filter was made through an 8-in. connection with a 12-in. pipe laid especially to bring filtered water to the filter plant from the force main leading from the city pumping station to the service reservoir, with a cross-connection also to the 16-in. return main, so that

water may be drawn from either one or the other of these two pipes as desired. The 12-in. pipe divides, on reaching the filter plant, an 8-in. branch, Plate XIII, entering the roughing-filter plant at the bottom of the channel which receives the rough-filtered water, with connections with the underdrain of each filter back of the effluent control valve, each of these connections being controlled by its own valve operated by a hand-wheel from the operating floor above. In addition, one other 8-in. branch is taken off from the 8-in. wash-water main and terminates in the sewer chamber of the central filter for use in flushing the sewer, if necessary.

In addition to the wash-water, provision is made for scouring the beds of the roughing filters with air admitted to the underdrains through 2-in. galvanized-iron pipes leading from the air receiver and provided with control valves and pressure gauges. The air pipes connect with the wash-water pipes in the bottom of the filtered-water effluent chamber of each filter between the wash-water control valve and the effluent underdrain.

Sewer Connection.—Under the regulating chambers of the roughing filters there is an 18-in. vitrified pipe sewer, Plates XIII and XVI, laid in concrete and extending to the river, a distance of approximately 1 000 ft. Into this sewer all parts of the plant can ultimately be drained.

The floor of the operating room of the roughing filters forms the roof over the various regulating chambers and the channel for the rough-filtered water; and the manholes providing for entrance into the various chambers are covered with light, 20-in., circular, cast-iron covers with ring-frames built into the concrete, manhole steps being provided in the walls.

General Water-Supply Pipes.—The water supply for mixing the coagulant solution, for supplying the steam-heating plant, the toilets, and the sand-washer plant of the slow filters, is taken from the 8-in. wash-water main in the effluent channel of the roughing filters by a vertical 3-in. galvanized-iron pipe rising above the ceiling of the operating room and extending lengthwise of the roughing filters to the sand-washer room in the second story of the building over the regulating chambers of the slow filters. The branches for the different fixtures are taken from this main line where required.

Operating Platform for Chemical Tanks.—From the floor of the operating room of the roughing filters, reinforced concrete stairways,

PLATE XVIII.
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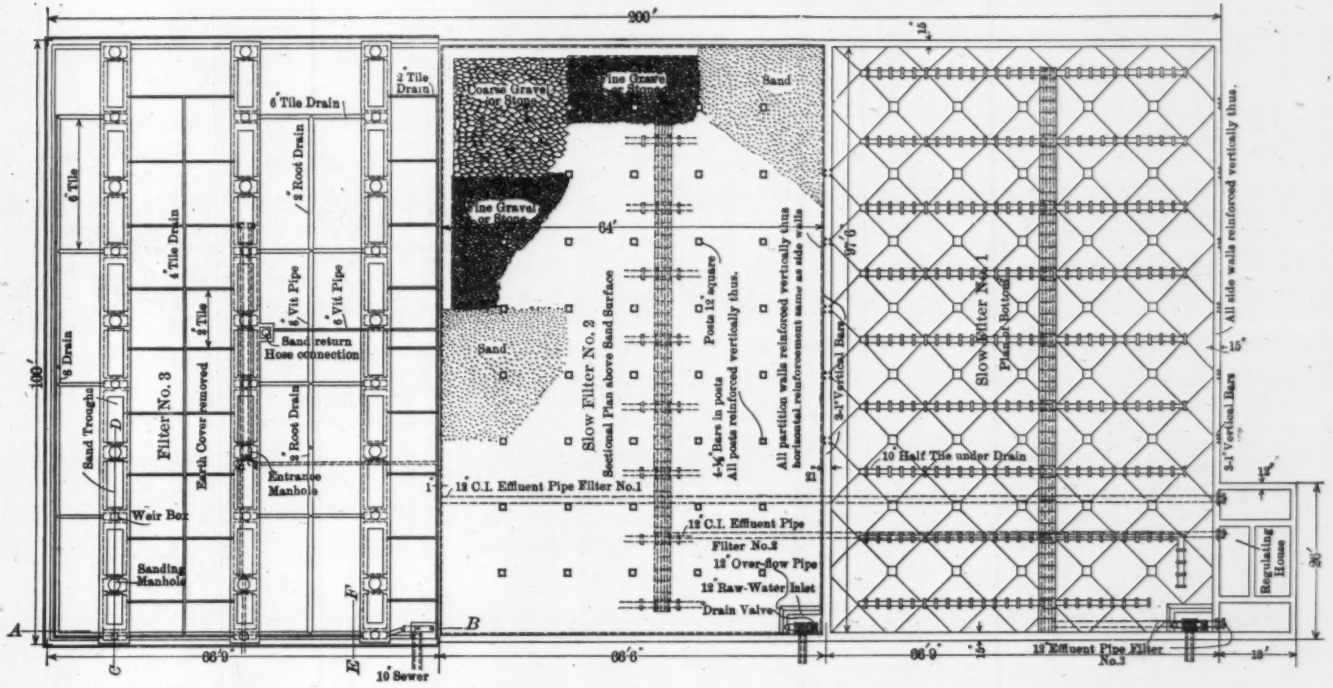
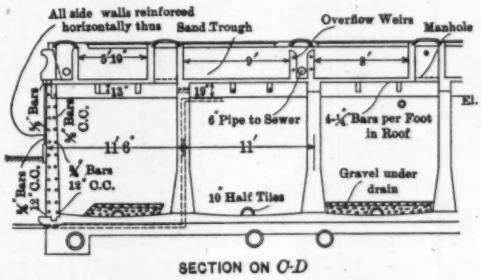
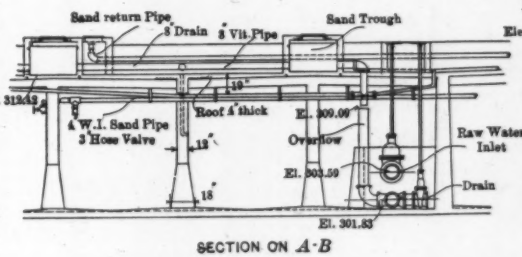
FUERTES ON
PURIFICATION OF WATER.

SLOW FILTERS

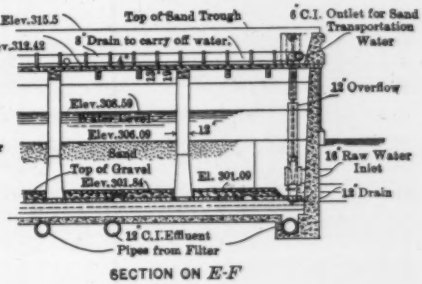




SECTION ON *C-D*



SECTION ON *A-B*



SECTION ON *E-F*

Plate XVI, lead up on either hand to the elevated platform in front of the coagulant-mixing tanks, and the lime-water mixing tanks. These are provided with iron-pipe railings, painted black and varnished.

Centrifugal Pump.—The centrifugal pump, Fig. 1, Plate XVII, which supplies the water through the measuring box to the lime-mixing tanks, has a 2-in. discharge pipe, and a 2-in. suction pipe, extending down into the channel containing the rough-filtered water, having on its lower end an elbow, short nipple, and gate-valve with extension stem reaching up above the floor of the operating room and there provided with a hand-wheel. Just below the floor level of the operating room a 2-in. horizontal branch extends through one of the roughing filters to the slow-filter operating room where a hose-valve for 2-in. suction hose is provided in order to permit of using the centrifugal pump to empty the regulating wells of the slow filters below the elevation to which they could be drained naturally.

Electric Motors.—Power to operate the machinery in the plant is derived from the electric current supplied by the York Haven Power Company, and is of the alternating-current type delivered at a voltage of 220 at the switch-board. The motors, Plate XV, of which there are two, manufactured by the Westinghouse Electric and Manufacturing Company, one of 5 h.p. and the other of 15 h.p., are of the alternating-current type, wound for 3-phase, 60-cycle, 200-volt terminal voltage, capable of standing an overload of 25% for 2 hours without injury, and provided with oil-immersion starting boxes. A marbleized-slate switch-board, mounted with volt meter, ampere meter, watt meters, circuit breakers, ground detectors, pilot lights, clock, and main and circuit switches for the different services required, stands in the machinery room.

Lighting.—All the filters and different parts of the plant are provided with electric light fixtures wherever light may be required, and all the rooms of the building containing the machinery and the regulating chambers of the different filters are also provided with gas jets.

Air Compressor.—The air compressor for supplying compressed air for scouring the filtering materials in the roughing filters, and for agitating the coagulant solution, stands between the two motors, and can be operated from either one. It is an Ingersoll-Rand compressor, known as the Imperial Type 11, and consists of two cylinders standing vertically, with single-acting plungers driven by crank shafts on opposite sides of a heavy fly-wheel which also serves as the belt pulley.

There is provided on the outlet from the compressor an unloading device, so-called, which, when the pressure reaches the maximum for which the machine is intended, allows the excess pressure to be relieved automatically.

The air compressor has a capacity of 39 cu. ft. of free air per min. at a speed of 200 rev. per min., and is capable of delivering the air at that rate under a pressure of 100 lb. per sq. in. The air cylinders are water-jacketed, with hooded ends, the cooling water being discharged into the trough of the filter below; the air valves are of the poppet type, and work vertically.

Air Receiver.—A 2-in. galvanized-iron discharge pipe, with a brass check-valve near the compressor, leads from the compressor to the air receiver, which is a vertical cylindrical tank, with domed ends, 5 ft. in diameter, and 12 ft. high. The shell is of flange-steel, is furnished with a manhole and a cast-iron base, and is proportioned to stand a working air pressure of 110 lb. per sq. in. and remain air-tight under that pressure. The receiver is provided with connections for the 2-in. galvanized-iron discharge pipe from the compressor and the 2-in. galvanized-iron air pipe leading from the receiver to the air wash-pipes of the roughing filters, and has also a 1¼-in drip at the bottom, provided with a double valve.

The receiver is provided with a safety valve set at 105 lb. per sq. in., a gauge to indicate the pressure in the receiver, and a 1¼-in. Foster reducing valve capable of discharging the entire contents of the receiver in 2 min. at a uniform pressure on the filter side of the valve of about 5 lb. per sq. in., the pressure of the receiver falling, at the same time, from 100 to about 10 lb. per sq. in.

The two motors, the air compressor, and the air receiver stand on foundations built monolithic with the roof of the west roughing filter, and no vibration is apparent when these machines are operating.

Countershaft.—The countershaft to which the motors and air compressor are belted is provided with a jaw coupler between the belt pulleys from the two motors, and is extended through the south wall of the machinery room into the regulating room of the roughing filters, Plates XIV and XVI, where additional countershafts are installed in order to reduce the speed as required for the operation of the grit elevator, and to increase the speed for the operation of the centrifugal pump supplying the water for the lime-water solution.

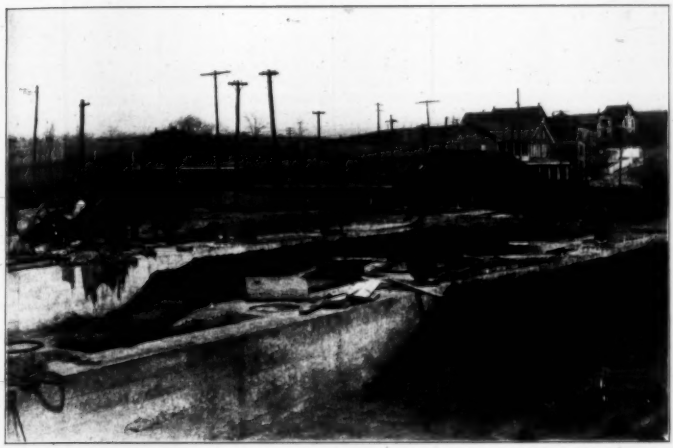


FIG. 1.—TOPS OF SLOW FILTERS DURING PLACING OF EARTH AND TOP-SOIL COVERING.

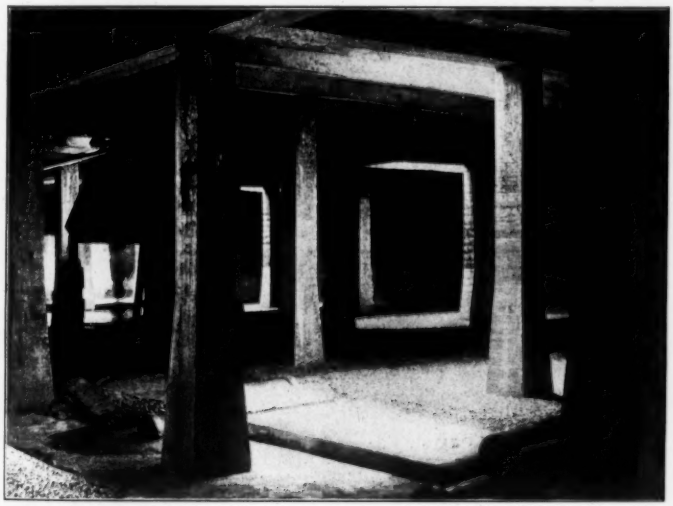


FIG. 2.—INTERIOR OF SLOW FILTER, SHOWING PLACING OF UNDERDRAINS AND GRAVEL LAYERS.

Superstructure.—The building covering the operating room of the roughing filters, the coagulant storage room, the machinery room, and the regulating room of the slow filters, Fig. 2, Plate XX, is L-shaped, one leg of which is 44 and the other 65 ft. long, measuring on the long sides, the two wings being, respectively, 21 and 26 ft. wide. The outer faces of the side walls are of red brick, white brick being used on the inside faces. The building is surmounted by a slate roof on a timber framing, with copper ridge, finials, gutters, and down-spouts, the down-spouts discharging into the troughs of the roughing filters, or into the deposit chamber, as necessary.

Sampling Devices.—In front of each roughing filter, and by the side of the entrance chamber for raw water, a small concrete bracket, Fig. 1, Plate XVII, and Fig. 1, Plate XXI, containing a copper funnel in the center, is fastened to the wall, and carries a nickel-plated goose-neck on the discharge pipe of a small rotary hand-pump, the mechanism of which is submerged in the water immediately below the shelf. The pumps are operated by extension shafts carrying nickel-plated hand-wheels. These pumps are used for obtaining samples of the raw water as it enters the plant and of the effluent from each roughing filter.

Slow Filters.

The rough-filtered water is conducted to the slow sand filters through a line of 16-in. wood-stave pipe with 12-in. connections to each slow filter, controlled by sluice-gates with stems extending to the tops of the filters under a manhole.

Area.—The slow filters, Plate XVIII, and Fig. 2, Plate XX, of which there are three, with a total net area of filtering surface of 0.4340 acre, lie side by side immediately north of the roughing filters. They are each 97 ft. 6 in. long, the two end filters being 64 ft. 7½ in., and the central filter, 64 ft. 9 in. wide, all dimensions being measured inside at the bottom, and all interior wall surfaces battering out toward the top 6 in. in their height. The thickness of the outside walls of the filters is 15 in. at the bottom, and 9 in. at the top; the thickness of the division walls is 21 in. at the bottom and 9 in. at the top. The walls are reinforced horizontally and vertically to take the necessary strains, and are built in sections ending against headers, with copper strips in all vertical joints to cut off any possible leakage at such points.

The floors of the filters are laid in blocks, and form inverted groined arches, with the columns supporting the roof standing on the high points.

Roof, Side Walls, and Floors.—The roof of each filter is supported on forty reinforced concrete posts, 12 in. square from the roof to a point 4¼ ft. from the bottom, battering out to a width of 18 in. on each side at the floor level. Longitudinally of each filter, reinforced concrete beams, 19 in. deep, run across the tops of the columns in alternate rows, while transversely of the filters, beams, 12 in. deep, run across the tops of all the columns in each row; on the intermediate lengthwise rows of columns the functions of beams for carrying the roof are performed by the deep sides of the troughs intended for the storage of sand removed from the filters during the periodical scrapings, the roof of the filter and the intersecting roof beams being tied to, and suspended from, the side walls of these troughs by vertical steel rods.

The reinforcement of the side walls of the filters consists of three vertical 1-in. bars, opposite each panel point of about 11 ft., running from top to bottom of the wall, the horizontal reinforcement consisting of ¾-in. bars, 12 in. from center to center, for the lower 8 ft. in depth, and ⅝-in. bars, 12 in. from center to center, for the remainder of the height, bars being used in both faces of each wall. The side walls rest in grooves formed along the outer edge of the floor slabs when the latter were laid. In laying the floor slabs, no particular precaution was taken to prevent leakage, further than to form grooves in each side of each slab as it was laid, allowing plenty of time for the concrete to harden and contract, before laying the next slab against it, and using a grillage of ½-in. bars, 4 ft. long, in the concrete of the floor under the bottom of each post supporting the roof. The grooves in the edges of the floor blocks were made V-shaped so that the settlement of any particular block would tend to wedge the joints tight.

Sand Troughs.—The sand troughs, Fig. 2, Plate XVII, and Fig. 1, Plate XIX, on top of the filters, of which there are three to each filter, are divided by cross partitions into boxes from 8 to 9 ft. long, separated at one end by weirs and at the other by manholes opening into the filter. The boxes are arranged so that the weirs of two adjacent boxes stand 12 in. apart, forming a channel crosswise of the sand box, from one end of which leads a 6-in. drain-tile connecting with a

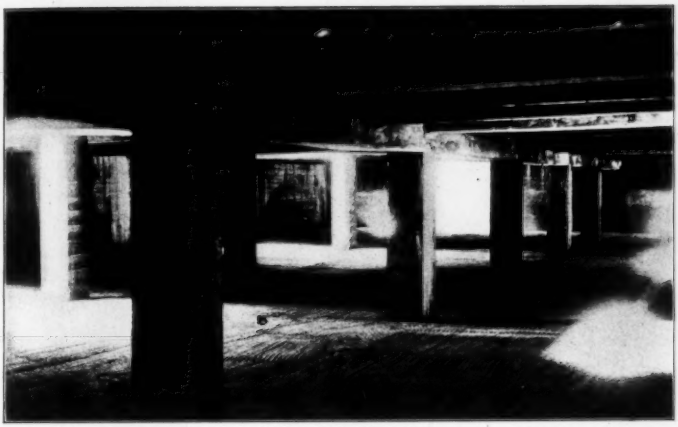


FIG. 1.—INTERIOR OF SLOW FILTER, SHOWING SAND IN PLACE.

FIG. 2.—TOPS OF SLOW FILTERS, SHOWING SAND TROUGHS.

system of drains running over the tops of the filters and discharging to the sewer through the overflow of each slow filter. The sand boxes have covers of ¼-in. checkered steel plates stiffened by angle irons riveted to the underside thereof, the cover-plates being in two halves for convenience in handling. Manhole heads and covers, 18 in. in diameter, are provided for the compartments containing the weirs, and 24-in. heads and covers for the manhole opening into the filter, except for the main entrance, for which there is a rectangular checkered steel cover, 2 by 4 ft. The sand boxes have 2-in. drain-tile connections at the bottom of each, and lead into the 4-in. tile drains between two adjacent rows of sand boxes; these connections allow the water to drain out of the sand when the box is filled from the sand-washing plant.

Frost Proofing.—Around the periphery of the plant a curb, 2.5 ft. high, of ornamental design was built on top of the roofs of the filters, and the entire roof, after the pipe drains were laid, was covered with earth, Fig. 1, Plate XIX, to a depth of 2.5 ft., the upper 9 in. being of top soil. The whole area was then raked over and sowed with grass seed.

Exterior Finish.—The visible exterior faces of the slow filter walls were divided into panels by pilasters moulded monolithic with the walls, the effect being carried around the roughing filters as a water-table from which to start the brickwork of the building containing the regulating rooms, storage rooms, etc.

Supply and Drain Valves.—In the southwest corner of each of the three slow filters is a compartment, Plate XVIII, the top edge of which is level with the surface of the sand. The rough-filtered water is delivered to the filters through these compartments. Drains for emptying the water from the surface of the filters, and overflow pipes connecting with a 12-in. drain, lead to the sewer. The drains from the sand troughs on the roof discharge vertically downward into the overflow pipe.

Slow-Filter Underdrains.—The underdrainage system of the slow filters consists, Plate XVIII, and Fig. 2, Plate XX, of lines of 10-in. half-tiles lying at right angles to the length of the filter in the center of each panel between the rows of posts, and connecting with a main underdrain running lengthwise of the filter in one of the panels adjacent to the center line; a 12-in. cast-iron pipe leads from

this central drain to the regulating house. The effluent control valves for all three filters are placed in a special house forming a part of the structure standing on the roughing filters. The half-tile drains were extended to a point about 4.5 ft. from the walls, all around, the ends being closed, and where they cross the central underdrain they were covered with concrete slabs forming part of the tight cover of the main underdrain. The vitrified hub and spigot tile drains were laid with open joints, and were surrounded with layers of broken stone, the first or bottom layer, about 6 in. thick, being composed of fragments of crushed sandstone the largest pieces of which were not more than 3 in. in diameter and the smallest not less than $\frac{3}{4}$ in. in diameter; the second layer, 3 in. thick, was formed of sandstone particles from $\frac{1}{4}$ to $\frac{3}{4}$ in. in diameter, from which the dust had been removed and in which there were few particles larger than $\frac{1}{2}$ in. in diameter; and the top layer, also 3 in. thick, was prepared from sandstone screenings, from which the dust had been removed and in which there were no particles larger than $\frac{1}{2}$ in. in diameter and very few larger than $\frac{1}{4}$ in. in diameter. The material was secured from a quarry at Marysville, on the Susquehanna, some miles above Harrisburg, and was in three sizes, $\frac{3}{4}$ in. to 3 in., $\frac{1}{4}$ to $\frac{3}{4}$ in., and screenings, the latter containing a considerable quantity of fine dust as well as particles up to $\frac{3}{4}$ in. in diameter. By proper sorting the only material requiring a second handling was the screenings, which, on sifting through $\frac{1}{4}$-in. and $\frac{3}{4}$-in. sieves, provided for all the fine material and for nearly one-half the $\frac{1}{4}$ to $\frac{3}{4}$-in. gravel required; but the weight of the wasted dust from the screenings exceeded, by approximately 25%, the weight of the useful materials secured. As received at the works, the materials were quite dirty, and a simple plan for washing out the stone dust was devised by the contractor's foreman. This consisted of a couple of oil barrels mounted tandem on horizontal axes slightly above the centers of the barrels, on a framework high enough to allow a dump-cart to stand thereunder. The two barrels were provided with a simple lever arrangement for turning them upside down. After being filled with water from a hydrant, they were suddenly upset on a cart containing, perhaps, $\frac{1}{2}$ cu. yd. of broken stone. This quick flush effectively washed the mud from the whole depth of stones in the cart, the surplus water draining out under the tailboard.

The gravel underdrains were not carried out to the side walls of the

FIG. 1.—SLOW-FILTER OPERATING ROOM.

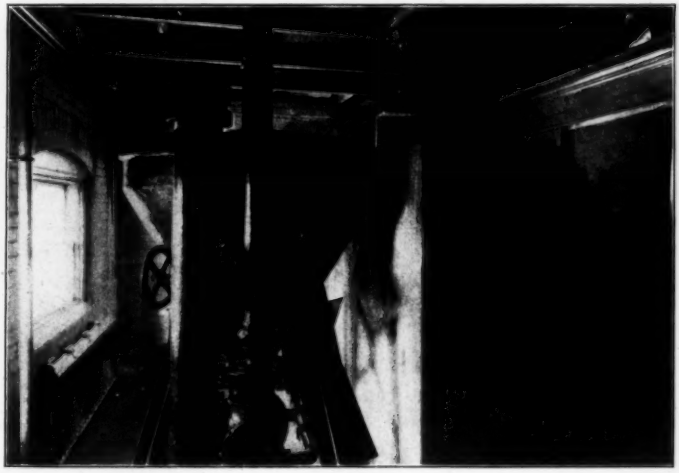


FIG. 2.—SAND WASHER.

filter, but were stopped about 2 ft. therefrom and banked up to the proper height, the finer layers of gravel covering the coarser layers on the slopes, and the top surface being brought to a uniform level.

Filter Sand.—The filter sand was secured from the Susquehanna River, and required no further preparation than screening, and, at times, the mixing of sands obtained from different localities.

The specifications for the sand were as follows:

Not more than 10% should be smaller than 0.29 mm. in diameter.
Not more than 5% should be smaller than 0.24 mm. in diameter.
At least 90% should be finer than 0.80 mm. in diameter.

Practically all the particles should be finer than 1 mm. in diameter, and the uniformity coefficient should be not greater than 1.6 to 1.8.

These specifications were written for the Susquehanna River sand with which the writer had had much experience; a modification of these requirements might be desirable for sands from other localities.

As to chemical composition, the Susquehanna River sands were satisfactory, containing less than 2% of lime and magnesia, and less than 5% of an aggregate amount of iron and aluminum in forms likely to disintegrate under the action of water or chemical changes produced by matters the water might contain when passing through the sand. The analyses of the samples of sand taken after the filters were ready to be put in operation are given in Table 1.

Filter No. 3 and practically all of Filter No. 2 were filled with sand obtained at Northumberland, from the West Branch of the Susquehanna, and required no further preparation than the careful selection of the barge loads as the sand was dredged, accepting those which would fill the requirements satisfactorily, as indicated by field tests conducted as the dredging proceeded, and rejecting those in which the sand was too fine or too dirty. The only difficulty experienced with this sand was the likelihood of its containing noticeable quantities of saw-dust, shreds of bark, and friable particles, but casting it by hand over screens having ¼-in. meshes removed the objectionable matters to a degree which rendered the sand satisfactory for filtration purposes.

Owing to the frequent changes in the conditions of flow in the Susquehanna, the character of the sand deposited is subject to variation, and, before a sufficient quantity had been secured, the Northumberland deposits had become too fine for use and it was necessary

to seek a coarser sand to mix with it in order to increase its effective size. An inspection of several deposits farther up the river finally disclosed one at Nesbit, from which much of the finer material had been washed, and by combining the mechanical analyses of this and the Northumberland sands, it was found that a satisfactory composite sand could be made by mixing the two in proper proportions.

TABLE 1.—MECHANICAL ANALYSES OF SAND IN SLOW FILTERS OF STEELTON FILTER PLANT.

Note.—In this table the figures in Column 1 give the depth from which the sample was taken, those in Column 2 refer to samples collected on the northeast corner of the filter, those in Column 3 to samples of the southeast corner, those in Column 4 to samples collected in the center of the filter, those in Column 5 to samples collected in the northwest corner of the filter, and those in Column 6 to samples collected in the southwest corner of the filter.

(1)	(2)		(3)		(4)		(5)		(6)	
	NORTHEAST CORNER.		SOUTHEAST CORNER.		CENTER.		NORTHWEST CORNER.		SOUTHWEST CORNER.	
Depth from which sample was taken.	Effective Size.	Uniformity Coefficient.	Effective Size.	Uniformity Coefficient.	Effective Size.	Uniformity Coefficient.	Effective Size.	Uniformity Coefficient.	Effective Size.	Uniformity Coefficient.
FILTER NO. 1.										
Top 0.5 ft.	0.32	1.7	0.32	1.5	0.32	1.7	0.31	1.5	0.31	2.0
0.5 to 1.5 ft.	0.30	1.8	0.30	1.6	0.31	1.7	0.31	1.6	0.31	1.9
1.5 to 3.0 ft.	0.29	1.9	0.32	1.5	0.32	1.5	0.30	1.6	0.32	2.0
3.0 to 4.0 ft.	0.31	1.5	0.33	1.3	0.29	1.7	0.31	1.6	0.32	1.8
FILTER NO. 2.										
Top 0.5 ft.	0.32	1.5	0.30	1.5	0.30	1.4	0.31	1.4	0.30	1.4
0.5 to 1.5 ft.	0.32	1.5	0.30	1.5	0.30	1.4	0.30	1.5	0.30	1.4
1.5 to 3.0 ft.	0.30	1.4	0.31	1.4	0.30	1.5	0.31	1.4	0.30	1.4
3.0 to 4.0 ft.	0.31	1.4	0.40	1.3	0.31	1.4	0.30	1.4	0.29	1.5
FILTER NO. 3.										
Top 0.5 ft.	0.31	1.6	0.30	1.8	0.31	1.8	0.31	1.7	Samples not taken.	
0.5 to 1.5 ft.	0.32	1.6	0.30	1.8	0.33	1.7	0.29	1.8		
1.5 to 3.0 ft.	0.32	1.6	0.29	1.7	0.36	1.5	0.30	1.8		
3.0 to 4.0 ft.	0.31	1.7	0.32	1.7	0.36	1.5	0.28	1.8		

As an example, Table 2 exhibits the analyses of the Northumberland and Nesbit sands on July 2d, 1908, at which time the mixing of the two sands in equal proportions produced the desired composite.

TABLE 2.

Sieve: Meshes, per inch.	Size of separation of sieves, in millimeters.	Nesbit sand. Percentage passing sieve.	Northumberland sand. Percentage passing sieve.	Composite sand. Percentage passing sieve.
20	1.02	100	100	100
30	0.57	64	65	65
50	0.38	8	23	13
60	0.29	2	14	7
80	0.21	0.3	0.6	0.5
Effective size		0.4 mm.	0.27 mm.	0.32 mm.
Uniformity coefficient		1.4	2.00	1.6

When Filter No. 1 was being filled, the Northumberland sand had an effective size of only about 0.27 or 0.28 mm. The two sands were mixed by dumping cart loads of each kind upon separate platforms on each side of a manhole, and casting simultaneously the correct relative numbers of shovelfuls from each pile upon a screen standing over the manhole. In passing through the screen and falling to the filter below, the two sands became thoroughly mixed, as will be seen from an inspection of the analyses in Table 1.

The sand was delivered to the filters in one-horse, two-wheeled carts. It was dumped through the manholes, and spread in three layers, care being taken not to compact it unevenly. The top surface, Fig. 1, Plate XX, was leveled off with straight-edges to a practically uniform elevation, the final operation being to rake the entire area over lightly with an iron rake. A typical mechanical analysis of the Northumberland sand, when running at its best, is given in Table 3.

The Northumberland sand, during the process of dredging from the river bottom, was quite thoroughly washed, and required no further treatment except screening through ¼-in. mesh screens to remove mussel shells, shreds of bark, etc., before being placed in the filters.

The total quantity of sand shipped to Steelton for the slow filters was 3 989 tons, in 107 car loads, and made, in the filters, 2 953 cu. yd.

In order to help out the contractor in the matter of securing this

sand, the writer kept an inspector at the sand dredges, whenever sand was being secured, during the entire period covering November and December, 1907, and January, April, May, June, and July, 1908; actual dredging operations, however, were limited to 92 days during this time.

TABLE 3.—MECHANICAL ANALYSIS OF SAMPLES OF NORTHUMBERLAND SAND, OCTOBER 31ST, 1907.

Size of sieve. Meshes, per linear inch.	Effective size of separation, in millimeters.	Weight of sand passing sieve, in pounds.	Percentage passing.
4	3.4	2.295	100
10	2.15	2.289	99.7
20	1.02	2.274	99
30	0.57	2.183	95
40	0.45	1.774	76
50	0.38	1.067	46
60	0.29	0.170	7.4
80	0.21	0.025	1.1
100	0.17	0.012	0.6
150	0.12	0.005	0.0

Note.—Effective size, 0.31 mm.; uniformity coefficient, 1.37: finer than 0.24 mm., 2.5%; finer than 1 mm., 99%; finer than 1.5 mm., 99.5 per cent.

The total cost of inspecting the sand, including the salary of the inspector, his board, and traveling expenses, was 9.5 cents per ton, or 13.0 cents per cu. yd. As the sand was running, a satisfactory indication of its character was obtained by a mechanical analysis of one sample to about each 30 cu. yd. dredged. The sand was sampled on the barge before unloading, each sample being a composite of samples collected at about four places from freshly excavated vertical faces in the sand as piled on the barge, and collected so as to be representative of the material for the full depth of the pile. These samples were then thoroughly mixed, the sample for analysis being taken therefrom, dried, and sifted through carefully rated screens.

Sand obtained in the river deposits in the neighborhood of Harrisburg could not have been used without expensive preparatory treatment to remove the fine material, as its effective size usually runs from about 0.23 to 0.24 mm. To remove some of the fine material, washing is necessary, the total waste ranging from about 10% to more than 50%, depending largely on the quantity of coal dust in the sand. The sand from Northumberland, being obtained from the West Branch above the junction of the North Branch, is free from coal, and is also coarser than that obtained farther down the river.

Effluent Control of Slow Filters.—The effluent pipes pass beneath the floors of the slow filters, and each terminates in its own regulating chamber with a sluice-gate controlled from a valve-stand and hand-wheel on the floor of the operating room. The rate of filtration of each filter is controlled by causing the filtered water to flow through a submerged orifice, Fig. 4, 8 in. in diameter in the end of an outlet pipe having a swivel-joint at the lower end, Plates XIII and XIV, and discharging through a gate-valve into a filtered-water well common to the three filters. The free or orifice end of the swivel-

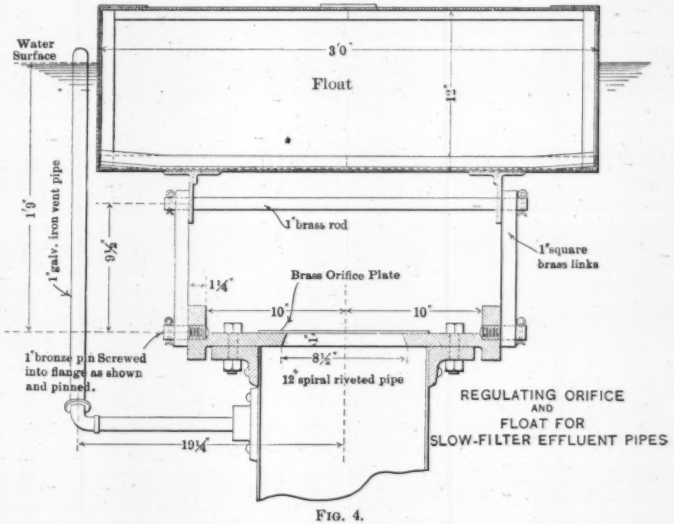


FIG. 4.

jointed pipe is suspended by brass links from a float resting on the surface of the filtered water in the effluent chamber of the filter; the center of the 8-in. orifice is adjusted to stand 1 ft. 9 in. below the surface of the water in the effluent well, at which depth, with a free discharge through the orifice into the atmosphere, the filter will deliver 1 500 000 gal. per day to the outer well. If the pumps in the Borough pumping station do not take the water away as fast as the three filters can furnish it, the water rises in the outer well, submerges the orifices, and cuts down the head by an amount just sufficient to keep up the supply, the plant being capable, therefore, of yielding the water to the

pumping station automatically at any rate necessary, up to the maximum for which the plant is designed, namely, 1 500 000 gal. per day per filter. In order to prevent any draft-tube effect in the effluent pipe, between the effluent well and the outer filtered-water well, a 2-in. galvanized-iron pipe is connected with the swivel-jointed pipe at a point below the orifice, the free end of the 2-in. pipe being adjusted so as to be always above the water level in the effluent well, whatever position the float may take; this assures a discharge into the atmosphere whenever the difference in level of the water between the effluent well and the filtered-water well of any filter exceeds 1 ft. 9 in.

Gauge Boards.—The rate of filtration and loss of head are indicated on scale boards, Fig. 1, Plate XXI, carried by bronze wires attached to floats in the different compartments of the regulating house, as described for the roughing filters. A sampling pump, similar to those for the roughing filters and the raw water, is provided for the effluent well of each slow filter.

The filtered water leaves the central filtered-water well through a 24-in. wood-stave pipe leading to the new pump-well at the pumping station, a sluice-gate being provided at each end of this pipe line.

Sand Washer.—The system of handling the sand when the slow filters require cleaning is not new in principle, but the details differ materially from those of other plants. Portable ejectors, supplied with water at about 100-lb. pressure from a line of 4-in. wrought-iron pipe, Plate XVIII and Fig. 2, Plate XIX, suspended from the ceiling of each slow filter, through 3-in. bronze hose-gates and 3-in. hose, lift the dirty sand and transport it to the sand-washer plant. The dirty sand, having been shoveled into piles in the filter, is cast into the hopper of the portable ejector, is picked up by the jet of water passing through the ejector, and delivered through a line of 4-in. wrought-iron pipe to the sand-washing hoppers, Plate XIV, and Fig. 2, Plate XXI, in the second story of the slow-filter regulating house. After passing through two hoppers the washed sand is forced by a jet in the bottom of the second hopper through a line of 4-in. wrought-iron pipe, resting on the roof of the three filters, to hose-valves from which lines of hose may be laid to discharge the returning clean sand into the compartments of the sand troughs running lengthwise of the tops of the filters. The operation is simple, and requires no special description or comment.

Heating.—The building covering the operating rooms of the rough-

ing and slow filters, as well as the machinery room and the coagulant-storage and sand-washer rooms, is heated by steam generated in a vertical tubular boiler standing in the second story of the slow-filter regulating house. The returns from the radiators are piped back to a trap in one corner of the slow-filter operating room, and discharge into the sewer. The provisions for heating required boiler capacity with sufficient heating surface and grate area to furnish steam to raise the temperature of 20 000 lb. of coagulant solution from 32° to 70° Fahr., in 1 hour. The boiler is designed to carry 100 lb. of steam, and is provided with a reducing valve, on the steam-heating connection, set at 5 lb. The coils and radiators are proportioned to maintain the machinery and operating rooms at a temperature of 70° and the coagulant-storage room and the sand-washer room at a temperature of 58° when the outdoor temperature is at zero. The toilet-room is provided with a wash-basin with cold-water connection and a steam connection for securing hot water, and a low-down tank closet.

Construction.

No special difficulties were encountered during construction, the bearing value of the foundations proving satisfactory for the relatively light loads they are required to carry. There was comparatively little ground-water in any of the work, although the permanent ground-water level was barely below the floor of the excavation for the slow filters; in fact, a modification of the design of the main underdrains of two of the filters was made necessary in order to keep the work out of the water. A considerable portion of the site of the slow filters was, in early days, a low swale which had been used subsequently as a dumping ground for slag, cinders, and ashes from the steel works, and it was owing to the presence of these materials, which had had such a long time to become thoroughly settled, that satisfactory foundations were secured without extra work.

Leakage.—Before the sand was placed in the slow filters they were filled with water from the street mains up to the level required for operation, the water from the street being then shut off and the drop in the surface level being observed. It was not expected that the filters would be perfectly tight, owing to the construction of the bottoms in blocks about 11 ft. square, with no special provision for stopping leakage between the blocks. The leakage from each of the

three filters was about the same, and aggregated about 0.66 of 1% of the nominal daily capacity of the plant. No tests to determine the amount of leakage have been made since the filters have been in operation, but it is believed that there has not been an increase.

Cost of the Plant.—The plans were completed in July, 1907, and, in response to advertisements, three bids were opened on August 7th, 1907, aggregating $71 522.85, $80 482.48, and $89 592.59, respectively. The contract, which was awarded August 14th, 1907, to the Bunting Construction Company, of Flushing, N. Y., stipulated that the work should be completed within 6 months from the date of the contract, or by February 14th, 1908. Various delays resulted, however, and it was not until the middle of September that the plant could be put in operation, the final estimate to the contractor being dated September 24th, 1908, and amounting to $70 730.87. This, however, did not represent the full cost of the work, as the installation of the centrifugal pumps, the changing of the suction pipes at the pumping station, and some repair work on the checkered steel covers on the slow filters, were done by the Water Board outside of the contract. In addition to the above items, there were the cost of the site and of engineering and incidental expenses, which were high, by reason of the fact that the time occupied by the contractor in the construction of the plant was more than twice that in which the work should have been completed under favorable conditions.

The following are the principal items making up the cost of the plant:

Excavation, grading, roadways, sodding, etc..	$3 936.88
Sewers and drains.....	2 314.67
Concrete, all classes.....	18 708.32
Reinforcing steel.....	3 475.80
Manhole heads and covers, indoors and outdoors.....	1 590.50
Cast-iron pipes and specials.....	3 956.27
Wood-stave pipes between filter plant and pumping station.....	9 034.21
Filter regulating devices, gauges, coagulant and lime-water apparatus.....	2 650.00
Superstructure over regulating houses, etc., including steam heating, lighting, and plumbing.....	7 900.00

Machinery, plant, motors, compressor, shafting, belting, air receiver, sand-washing plant, etc.	3 750.00
Gate and sluice valves	1 485.00
Filtering materials, including underdrains	10 811.34
Extra work	1 117.88
	$70 730.87

Of the above amount, $57 000.00 is the cost of the purification works, the remainder, $13 700.00, being the cost of the pipes to the pumping station, the sewer to the river and other details growing out of local conditions.

Operation.

The plant was put in service on September 12th, 1908, but was shut down later in the day, and was started again on September 16th, since which date it has furnished filtered water continuously.

The regular force required for operation consists of a general superintendent and two filter attendants, the latter working on 12-hour shifts. When the slow filters require scraping an additional force is taken on temporarily, the men working under the direction of the general superintendent.

The permit for the construction of the plant, issued by the Pennsylvania State Department of Health, provided that the filters should be operated for a considerable period under the direction of its designer, and in compliance with this condition the writer instructed the superintendent and attendants in their duties, provided specific instructions regarding the manipulation of the roughing and slow filters, and the handling of the coagulant, and has advised with reference to special features of the operation whenever necessary.

From September 16th, 1908, until January 9th, 1909, no coagulant was used in connection with the operation of the roughing filters, the raw water being sufficiently free from turbidity to yield a satisfactory water without coagulant.

Quantity of Coagulant Required.—The general instructions given to the superintendent called for the application of a coagulant to the raw water whenever its turbidity exceeded 50 parts per million, and the continuance of its use for much lower turbidities whenever the

last bacterial analyses showed 5 000 or more bacteria per cu. cm. in the raw water, or in case the effluent from the slow filters showed objectionable color. It is the intention, in dosing with coagulant, to use a sufficient quantity to produce an effluent from the roughing filters having a turbidity of not more than 10% of that of the applied water, and not, in any case, more than 25 parts per million, regardless of the turbidity of the raw water. It has also been found desirable, particularly during the winter and the cold spring weather, to manipulate the dose of coagulant so as to produce an effluent from the rough-

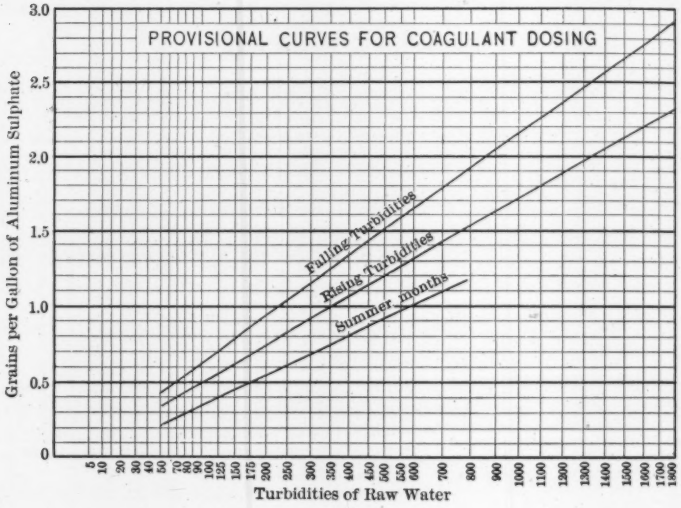


FIG. 5.

ing filters having a turbidity of practically zero, as by this means the runs of the slow filters are very greatly lengthened and the cost of operation is not increased by the use of the comparatively small additional amount of coagulant necessary to secure this result.

The quantity of coagulant to be used is determined by the superintendent from a diagram, Fig. 5, prepared by the writer from data secured originally during the operation of the Harrisburg Testing Station, in 1902 and 1903, but modified somewhat to take into account the difference in the character of the filtering materials, and the slightly different character of the raw water at the Steelton intake; the

diagram may require further modification as additional knowledge is gained during the operation of the plant. The dose of coagulant indicated by the lowest curve is for application during the summer when the turbidities do not rise very high, or very suddenly. The dosing indicated by the middle curve is required when the turbidity is rising rapidly, during the first parts of the floods in the fall, winter and spring; the dosing indicated by the upper curve is required when the turbidity is falling, after the passage of a flood. It will be noticed that for equal turbidities more coagulant is required when the turbidities are falling than when they are rising, on account of the finer character of the turbidity and the persistence of high numbers of bacteria in the Susquehanna after the turbidities have begun to fall. The superintendent must use judgment in determining the proper dose of coagulant. The character of the turbidity of the river water varies so greatly and changes so quickly that a dose satisfactory with a given clay turbidity, say, of 500 parts per million, might prove altogether too much for an equal turbidity from the North Branch. At times, also, particularly after a protracted season of low water, a turbidity caused largely by the fine shreds of vegetation torn and scoured loose from the rocky river bottom is particularly difficult to handle by reason of its high clogging value when it becomes matted upon the filter surface. In order to keep the plant in efficient operation, therefore, the superintendent must watch the effect of his dosing and reduce the quantities if he finds the loss of head increasing on the roughing filters too rapidly, or increase the quantity if the conditions require it.

When to Wash the Roughing Filters.—If, when coagulant is being applied to the raw water in the proper amount, the effluent of a roughing filter shows objectionable turbidity, indicating the passage of coagulated material through the filter bed, the filter is immediately washed; otherwise, it is left in operation until the maximum allowed loss of head, about 2.5 ft., is reached.

This method of operation is based on the theory, which seems to be supported by considerable evidence gained both at this plant and at the Harrisburg Testing Station, that these coarse-grained filters act very much as settling basins with bottom areas of great extent. The mud carried into the filter by the entering water is deposited on the granules of the filter bed through practically its entire depth, and one

of the requirements in controlling their operation is to prevent the washing of this sediment through the filters by pushing their operation too rapidly, or by using too great losses of head, or by suddenly increasing the rates of filtration. To prevent most of these evils the mechanisms at the plant limit the loss of head to about 2.5 ft.

Regulation of Coagulant Dosing.—The quantity of solution required to supply a given dose of coagulant depends on the quantity of water being filtered and the percentage strength of the solution.

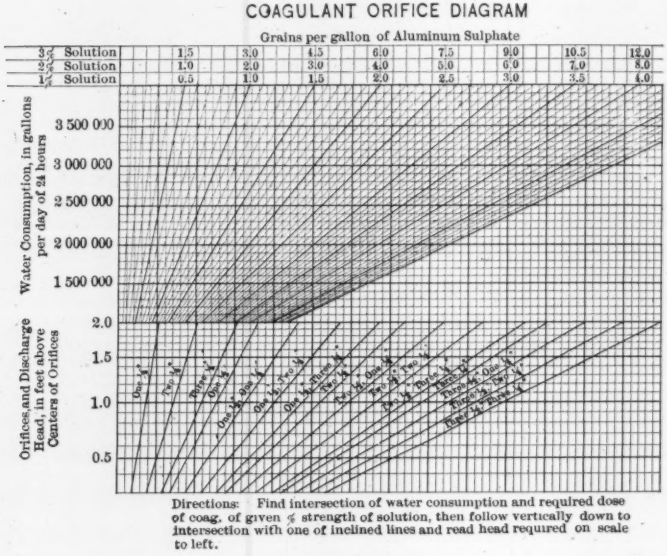


FIG. 6.

Knowing these, the requisite quantity of solution is measured out by continuous discharge through orifices in the glass plate forming the front of the measuring box, the size and number of orifices, as well as the head required to give the necessary dose, in grains per gallon, being ascertained by the superintendent from a diagram, Fig. 6, prepared for his use. The lines indicating the orifices to be used were placed on the diagram in accordance with the measured discharge of each orifice.

Use of Lime-Water.—During floods the alkalinity of the river water

falls sometimes too low to permit of the decomposition of the requisite dose of aluminum sulphate, and provisions are made to add, at such times, sufficient milk of lime to supply the deficiency in alkalinity. When lime must be used, enough is added to neutralize the free CO_2 in the river water, combine with the aluminum sulphate, and leave a residual alkalinity in the water of about 6 parts per million. The free CO_2 and alkalinity are determined by the methods recommended by the Committee on Standard Methods of Water Analysis, of the American Public Health Association. Knowing the dose of aluminum sulphate being used, and having ascertained the alkalinity of the raw water, and the amount of free CO_2 carried, reference to a diagram including all these data will give the quantity of lime required, if any.

Regulation of Lime Dosing.—The quantity of water required to add this dose of lime to the raw water in a saturated solution is regulated at the lime-water measuring box in the manner in which the coagulant dose is regulated, a diagram similar to the coagulant-dosing diagram being supplied for the lime-water. The amount of lime to be slaked daily to produce the required quantity of lime-water, for different dosings and different daily consumptions of water, is determined by the superintendent from another diagram, small quantities being slaked from time to time during the day and discharged as milk of lime through the funnel and pipe leading to the bottom of the lime-mixing tanks. The strength of the lime-water solution is to be tested from time to time by the usual simple chemical test, and it should be kept up to full strength by the attendant.

Another diagram gives the number of pounds of aluminum sulphate required for making solutions of different percentage strength, the capacity of the tanks having been calibrated for the purpose.

Simplicity of the Dosing System.—No calculations being required on the part of the superintendent during the operation of the plant, the rapid changing of the dosing, when required, can be effected with accuracy and without the likelihood of errors from faulty calculations.

The lime-water and the coagulant solution are admitted to the water practically together. The conditions requiring the use of lime being comparatively rare no great pains have been taken to go into refinements in the use of lime that would be desirable under conditions requiring its more continuous use.

Reduction of Alkalinity by Coagulant.—As to the proportion of the coagulant rendered inactive through its absorption by suspended matters in the water, sufficient data have not yet been obtained at the Steelton plant to speak authoritatively, but it is believed, from present indications, that the reduction of alkalinity of the raw water, instead of the theoretical 8.2, will probably be about 6 parts per million, per grain of coagulant used, as observed at the Harrisburg Filter Plant during the past three years and at the Harrisburg Testing Station in 1902 and 1903.

OPERATION OF ROUGHING FILTERS.

Operation with Respect to Lengths of Runs.—While the roughing filters must be operated in a manner to secure an effluent sufficiently free from turbidity and bacteria to be handled satisfactorily by the slow filters, their operation must be controlled in a manner to secure sufficiently long periods of operation between washings, otherwise the plant could not be kept going during periods of very turbid water. A little experience is required to enable this to be done, but it presents no practical difficulty with any range of conditions thus far encountered on the Susquehanna at Harrisburg and Steelton. The highest turbidity experienced since the plant has been in operation occurred in the forenoon of January 26th, reaching 1 600 parts per million, remaining at that figure for about 5 hours, and dropping by steps to 500 parts during the next 12 hours. The shortest run during this period was about 2½ hours, which occurred after the turbidity had dropped to about 700 parts. The first turbid water the plant was required to handle occurred about January 8th, at a time when the writer was not able to be present, and, owing to the deterioration of certain of the silica-turbidity standards, the turbidities of the raw water and of the effluents from the roughing filters were incorrectly read, the actual turbidities being much higher than the figures recorded. As a result, the quantity of coagulant used was entirely too small, and a considerable number of bacteria passed through both the roughing filters and the final filters.

Records of Operation.—Since January 23d, 1909, with but very few exceptions, the coagulant has been properly applied, and the satisfactory results obtained in the removal of turbidity are exhibited in Table 4, which gives the hourly records of the operation of Roughing Filter No. 1, through runs Nos. 58 to 81, January 26th to 31st, 1909, inclusive.

TABLE 4.—Hourly Records of Operation of Roughing Filter No. 1 Through Runs Nos. 58 to 81, January 26th to 31st, 1909.

Number of run.	Lengths of run.		Date.		Turbidities, in parts per million.			Coagulant used, in grains per gallon.
	Hours.	Minutes.	Day.	Hour.	Raw water.	Rough-filtered water.	Percentage of removal.	
58	9	58	Jan. 26	6 A. M.	80	1	98.8	0.42
				7 "	800	1	99.9	0.42
				8 "	800	1	99.9	0.92
59	4	16		9 "	800	1	99.9	1.84
				10 "	1 600	1	99.9	2.22
				11 "	1 600	1	99.9	2.22
				12 M.	1 600	1	99.9	2.22
				1 P. M.	1 600	1	99.9	2.22
60	3	02		2 "	1 600	1	99.9	2.22
				3 "	900	1	99.9	2.00
				4 "	900	1	99.9	2.00
61	2	31		5 "	900	1	99.9	2.00
				6 "	900	1	99.9	2.00
				7 "	860	1	99.9	2.00
62	2	26		8 "	700	1	99.8	1.50
				9 "	700	1	99.8	1.50
63	4	35		10 "	700	1	99.8	1.50
				11 "	700	1	99.8	1.00
				12 "	500	1	99.8	0.75
			Jan. 27	1 A. M.	500	1	99.8	0.75
				2 "	530	2	99.6	0.75
64	2	58		3 "	500	2	99.6	0.75
				4 "	500	2	99.6	0.75
				5 "	500	1	99.8	0.75
65	2	11		6 "	500	1	99.8	0.75
				7 "	600	2	99.7	0.63
				8 "	750	2	99.7	0.63
66	3	18		9 "	800	3	99.6	0.63
				10 "	800	2	99.8	0.63
				11 "	750	2	99.7	0.63
				12 M.	750	2	99.7	0.63
67	3	17		1 P. M.	700	2	99.7	0.63
				2 "	600	3	99.5	0.63
				3 "	550	2	99.6	0.63
68	4	15		4 "	550	2	99.6	0.63
				5 "	550	2	99.6	0.63
				6 "	550	3	99.4	0.63
				7 "	550	3	99.4	0.63
69	3	53		8 "	550	3	99.4	0.63
				9 "	550	3	99.4	0.63
				10 "	550	3	99.4	0.80
				11 "	500	10	98.0	1.95
				12 "	500	2	99.6	1.95
70	3	00	Jan. 28	1 A. M.	500	1	99.8	1.50
				2 "	500	1	99.8	1.50
				3 "	500	1	99.8	1.50
71	4	03		4 "	500	1	99.8	1.50
				5 "	500	1	99.8	1.50
				6 "	500	1	99.8	1.50
				7 "	500	1	99.8	1.30

TABLE 4.—(*Continued.*)

Number of run.	Lengths of run.		Date.		Turbidities, in parts per million.			Coagulant used, in grains per gallon.
	Hours.	Minutes.	Day.	Hour.	Raw water.	Rough-filtered water.	Percentage of removal.	
72	4	04	Jan. 28.	8 A. M.	500	1	99.8	1.30
				9 "	500	1	99.8	1.30
				10 "	500	1	99.8	1.30
				11 "	500	1	99.8	1.22
73	4	56		12 M.	450	1	99.7	1.22
				1 P. M.	500	1	99.8	1.22
				2 "	500	1	99.8	1.22
				3 "	500	1	99.8	1.22
				4 "	500	1	99.8	1.22
				5 "	500	1	99.8	1.22
74	5	13		6 "	500	1	99.8	1.22
				7 "	500	1	99.8	1.30
				8 "	450	1	99.7	1.30
				9 "	450	1	99.7	1.30
				10 "	450	1	99.7	1.30
75	5	32		11 "	450	1	99.7	1.30
				12 "	450	1	99.7	1.30
			Jan. 29	1 A. M.	450	1	99.7	1.30
				2 "	450	1	99.7	1.30
				3 "	450	1	99.7	1.40
				4 "	450	1	99.7	1.40
76	5	19		5 "	450	1	99.7	1.40
				6 "	450	1	99.7	1.40
				7 "	400	1	99.7	1.40
				8 "	400	1	99.7	1.40
				9 "	300	1	99.7	1.40
77	6	50		10 "	300	0	100	1.30
				11 "	300	0	100	1.30
				12 M.	300	0	100	1.30
				1 P. M.	300	0	100	1.30
				2 "	275	0	100	1.30
				3 "	275	0	100	1.20
				4 "	275	0	100	1.20
				5 "	275	0	100	1.20
78	8	05		6 "	250	0	100	1.12
				7 "	250	0	100	1.12
				8 "	250	0	100	1.12
				9 "	250	0	100	1.12
				10 "	250	0	100	1.12
				11 "	250	0	100	1.12
				12 "	250	0	100	1.12
			Jan. 30	1 A. M.	250	0	100	1.12
79	10	59		2 "	250	0	100	1.12
				3 "	250	0	100	1.12
				4 "	250	0	100	1.12
				5 "	250	0	100	1.12
				6 "	250	0	100	1.12
				7 "	250	0	100	1.12
				8 "	250	0	100	1.12
				9 "	250	0	100	1.12
				10 "	250	0	100	1.12
				11 "	250	0	100	1.12
				12 M.	250	0	100	1.12
80	17	00		1 P. M.	200	0	100	1.12
				2 "	200	0	100	1.12
				3 "	200	0	100	0.99

TABLE 4.—(*Continued.*)

Number of run.	Lengths of run.		Date.		Turbidities, in parts per million.			Coagulant used, in grains per gallon.
	Hours.	Minutes.	Day.	Hour.	Raw water.	Rough-filtered water.	Percentage of removal.	
80	17	00	Jan. 30	4 P. M.	200	0	100	0.99
				5 "	200	0	100	0.99
				6 "	200	0	100	0.99
				7 "	200	0	100	0.99
				8 "	200	0	100	0.99
				9 "	200	0	100	0.99
				10 "	200	0	100	0.99
				11 "	200	0	100	0.99
				12 "	200	0	100	0.99
				1 A. M.	200	0	100	0.99
				2 "	200	0	100	0.99
				3 "	200	0	100	0.99
				4 "	150	0	100	0.99
				5 "	150	0	100	0.99
81	13	40		6 "	150	0	100	0.99
				7 "	150	0	100	0.99
				8 "	100	0	100	0.63
				9 "	100	0	100	0.63
				10 "	100	0	100	0.63
				11 "	100	0	100	0.63
				12 M.	75	0	100	0.55
				1 P. M.	75	0	100	0.55
				2 "	75	0	100	0.55

Four of the runs included in Table 4 were too short, and, with the experience since gained in handling the plant, particularly the coagulant, could have been extended by an hour or two without difficulty.

The complete records of the operation of Roughing Filter No. 1 by runs, from September 12th, 1908, to February 3d, 1909, are given in Table 5.

Analyses.—There is no laboratory for chemical and bacteriological work at the plant, but an arrangement has been made by which samples of the raw water, the rough-filtered water, the final-filtered water, and water secured from a faucet in the borough, are collected and examined by the chemist and bacteriologist of the Harrisburg Filter Plant once each week, and at such other times as desired. Two plates, for total numbers of bacteria, are prepared from each sample, and presumptive tests are made for the presence of *B. Coli* in the raw- and filtered-water samples, five 1-cu. cm. sowings usually being made of each sample; the tests for turbidity, free CO_2, and alkalinity are made at the plant by the superintendent, who is a skilled chemist.

TABLE 5.—(*Continued.*)

Run No.	From:	To:	Time in operation: Hours.	Time in operation: Minutes.	Loss of head: Initial.	Loss of head: Final.	Average rate of operation, in gallons daily.	Millions of gallons per acre per day.	Silica turbidity, in parts per million. Raw water.	Silica turbidity, in parts per million. Effluent.	Silica turbidity, in parts per million. Percentage removed.	Bacteria per cubic centimeter. Raw water.	Bacteria per cubic centimeter. Effluent.	Bacteria per cubic centimeter. Percentage removed.	Coagulant, in grains per gallon (average).
36	Jan. 10	Jan. 11	6	02	0.50	1.60	780 000	84.3	400	19.0	95	.....	....	..	....
37	" 11	" 11	4	21	0.50	1.58	890 000	102.0	400	20.0	95	18 200	6 000	68	0
38	" 11	" 11	5	18	0.48	1.95	810 000	92.9	400	28.0	93	.....	....	..	0
39	" 11	" 11	4	00	0.52	1.96	810 000	92.9	400	30.0	92	.....	....	..	0
40	" 11	" 11	3	11	0.55	1.95	880 000	100.8	400	30.0	92	.....	....	..	0
41	" 11	" 12	2	58	0.83	1.65	860 000	98.5	450	19.0	96	.....	....	..	0.8
42	" 12	" 12	3	40	0.60	2.15	750 000	84.9	450	13.0	97	.....	....	..	0.9
43	" 12	" 12	4	09	0.45	2.15	800 000	91.7	500	9.0	98	.....	....	..	0.9
44	" 12	" 12	2	44	0.60	1.55	890 000	102.0	500	6.0	98	.....	....	..	0.9
45	" 12	" 12	2	38	0.65	1.15	820 000	94.0	500	5.0	99	.....	....	..	0.9
46	" 12	" 12	3	43	0.50	1.90	830 000	95.0	500	6.0	98	.....	....	..	0.9
47	" 12	" 12	4	02	0.60	1.87	720 000	82.5	500	9.0	98	.....	....	..	0.9
48	" 12	" 13	8	01	0.74	1.55	770 000	88.2	450	7.0	98	.....	....	..	0.9
49	" 13	" 14	27	57	0.45	1.55	760 000	87.0	450	24.0	95	.....	....	..	0.3
50	" 14	" 15	12	05	0.52	1.74	770 000	88.2	300	5.0	98	.....	....	..	0.40
51	" 15	" 15	9	01	0.60	1.60	790 000	90.7	150	4.0	97	32 000	6 200	80	0.38
52	" 15	" 15	9	45	0.45	1.75	790 000	90.7	150	4.0	97	.....	....	..	0.33
53	" 15	" 16	4	40	0.55	1.22	760 000	87.1	125	4.0	97	.....	....	..	0.25
54	" 16	" 20	83	39	0.57	1.46	750 000	86.0	50	4.0	92	.....	....	..	....
55	" 20	" 23	68	14	0.48	1.60	720 000	82.5	20	3.0	85	5 975	1 445	76	0.10
56	" 23	" 24	17	38	0.62	1.45	760 000	83.7	15	1.0	93	.....	....	..	0.25
57	" 24	" 25	19	13	0.50	1.29	660 000	75.7	40	2.0	95	.....	....	..	0.25
58	" 25	" 26	9	58	0.38	1.75	630 000	72.2	560	1.0	99	.....	....	..	0.45
59	" 26	" 26	4	16	0.40	1.60	680 000	78.0	1 440	1.0	99	.....	....	..	2.07
60	" 26	" 26	3	02	0.40	2.10	780 000	89.4	1 200	1.0	99	.....	....	..	2.08
61	" 26	" 26	2	31	0.95	2.25	1 000 000	114.7	900	1.0	99	.....	....	..	2.00
62	" 26	" 26	2	26	0.95	1.96	840 000	96.2	700	1.0	99	.....	....	..	1.50
63	" 26	" 27	4	35	0.54	2.25	650 000	75.7	600	1.0	99	.....	....	..	1.58
64	" 27	" 27	2	58	0.57	1.98	860 000	98.5	500	2.0	99	.....	....	..	0.75
65	" 27	" 27	2	11	0.45	2.32	910 000	104.3	650	2.0	99	13 160	1 025	92	0.67
66	" 27	" 27	3	18	0.44	2.20	750 000	86.1	750	2.0	99	.....	....	..	0.63
67	" 27	" 27	3	17	0.40	1.75	810 000	92.9	600	2.0	99	.....	....	..	0.63
68	" 27	" 27	4	15	0.40	1.89	760 000	87.1	550	3.0	99	.....	....	..	0.63
69	" 27	" 28	3	53	0.45	2.29	830 000	95.2	525	4.0	99	.....	....	..	1.19

TABLE 5.—SUMMARY OF RESULTS OF OPERATION OF ROUGHING FILTER NO. 1, BY RUNS.

Run No.	From:	To:	Time in operation: Hours.	Minutes.	Loss of head: Initial.	Final.	Average rate of operation, in gallons daily.	Millions of gallons per acre per day.	Silica turbidity, in parts per million. Raw water.	Effluent.	Percentage removed.	Bacteria per cubic centimeter. Raw water.	Effluent.	Percentage removed.	Coagulant, in grains per gallon (average).
1	Sept. 12	Oct. 5	570	43	0.35	1.76	710 000	81.4	6	....	...	.....	....	..	....
2	Oct. 5	" 19	334	00	0.53	1.18	707 000	81.0	2	0.5	75	.....	....	..	....
3	" 19	" 29	245	46	0.45	1.98	692 000	79.3	2	0.3	87	.....	....	..	....
4	" 29	Nov. 3	125	03	0.45	1.70	700 000	80.3	7	0.0	100	.....	....	..	....
5	Nov. 3	" 8	125	37	0.53	1.18	690 000	79.0	3	0.0	100	80	....	..	....
6	" 8	" 12	90	07	0.45	0.75	720 000	82.5	2	0.0	100	.....	....	..	....
7	" 12	" 16	95	23	0.54	0.85	685 000	78.5	2	0.0	100	.....	....	..	....
8	" 16	" 20	91	46	0.55	1.04	760 000	87.1	2	0.0	100	119	76	39	....
9	" 20	" 28	186	41	0.51	1.32	700 000	80.3	2	0.0	100	214	12	92	....
10	" 28	Dec. 4	161	00	0.45	1.68	770 000	88.2	1	0.0	100	.....	....	..	....
11	Dec. 4	" 8	102	55	0.60	1.08	770 000	88.2	1	0.0	100	408	310	24	....
12	" 8	" 14	136	31	0.88	1.06	800 000	91.7	2	0.0	100	220	125	43	....
13	" 14	" 18	94	41	0.58	1.04	710 000	81.3	5	1.0	80	1 680	680	60	....
14	" 18	" 20	50	10	0.40	1.60	690 000	79.0	7	1.0	87	.....	....	..	....
15	" 20	" 22	51	09	0.38	1.50	705 000	80.8	9	3.0	66	4 425	800	80	....
16	" 22	" 24	39	02	0.60	1.61	750 000	86.0	15	3.0	80	.....	....	..	....
17	" 24	" 28	94	56	0.65	1.30	760 000	67.1	7	3.0	60	.....	....	..	....
18	" 28	Jan. 1	87	17	0.56	0.92	750 000	86.0	4	1.0	75	.....	....	..	....
19	Jan. 1	" 3	53	54	0.75	1.48	750 000	86.0	2	1.0	66	5 800	1 125	80	....
20	" 3	" 6	76	52	0.62	1.58	790 000	90.6	4	1.0	75	.....	....	..	....
21	" 6	" 7	10	16	0.58	1.04	840 000	96.3	10	4.0	60	.....	....	..	....
22	" 7	" 7	13	38	0.42	1.02	810 000	92.9	100	4.0	96	.....	....	..	....
23	" 7	" 8	6	45	0.60	1.30	870 000	99.7	225	8.0	96	.....	....	..	....
24	" 8	" 8	5	39	0.48	1.30	860 000	98.6	300	8.0	97	.....	....	..	....
25	" 8	" 8	9	57	0.48	1.56	760 000	87.1	350	16.0	96	.....	....	..	....
26	" 8	" 9	7	56	0.62	1.55	890 000	96.1	400	14.0	97	.....	....	..	....
27	" 9	" 9	4	05	0.55	1.10	850 000	97.4	425	19.0	95	.....	....	..	....
28	" 9	" 9	3	32	0.57	1.45	860 000	98.6	450	21.0	93	.....	....	..	1.3
29	" 9	" 9	4	22	0.60	1.53	840 000	96.3	500	3.0	99	.....	....	..	0.6
30	" 9	" 9	3	57	0.67	1.63	870 000	99.7	550	3.0	99	.....	....	..	0.35
31	" 9	" 10	3	37	0.58	1.42	850 000	97.4	450	6.0	98	.....	....	..	0.3
32	" 10	" 10	5	07	0.54	1.80	810 000	92.9	450	15.0	96	.....	....	..	0.3
33	" 10	" 10	4	57	0.35	1.50	850 000	97.4	400	33.0	92	.....	....	..	0.3
34	" 10	" 10	3	51	0.56	1.75	890 000	102.0	400	13.0	97	.....	....	..	0.3
35	" 10	" 10	5	14	0.54	1.70	800 000	91.7	400	18.0	96	.....	....	..	0.3

TABLE 5.—(*Continued.*)

Run No.	From:	To:	Time in operation:		Loss of head:		Average rate of operation, in gallons daily.	Millions of gallons per acre per day.	Silica turbidity, in parts per million.			Bacteria per cubic centimeter.			Coagulant, in grains per gallon (average).
			Hours.	Minutes.	Initial.	Final.			Raw water.	Effluent.	Percentage removed.	Raw water.	Effluent.	Percentage removed.	
70	Jan. 28	Jan. 28	3	00	0.60	2.17	760 000	87.1	500	1.0	99	.....	....	..	1.50
71	" 28	" 28	4	03	0.40	2.20	760 000	87.1	500	1.0	99	.....	....	..	1.45
72	" 28	" 28	4	04	0.45	2.20	780 000	89.5	500	1.0	99	.....	....	..	1.28
73	" 28	" 28	4	56	0.60	2.16	780 000	89.5	500	1.0	99	.....	....	..	1.22
74	" 28	" 28	5	13	0.50	1.82	750 000	86.1	450	1.0	99	.....	....	..	1.28
75	" 28	" 29	3	32	0.40	2.11	720 000	82.6	450	1.0	99	.....	....	..	1.33
76	" 29	" 29	5	19	0.57	1.99	740 000	84.9	300	0.0	100	.....	....	..	1.44
77	" 29	" 29	6	50	0.57	1.99	700 000	80.3	250	0.0	100	.....	....	..	1.26
78	" 29	" 30	8	05	0.40	1.88	710 000	81.4	250	0.0	100	.....	....	..	1.12
79	" 30	" 30	10	59	0.44	1.95	580 000	66.6	250	0.0	100	.....	....	..	1.12
80	" 30	" 31	17	00	0.44	1.86	560 000	64.2	250	0.0	100	.....	....	..	1.00
81	" 31	" 31	13	40	0.65	1.43	740 000	84.9	100	0.0	100	.....	....	..	0.65
82	" 31	Feb. 1	10	38	0.55	1.40	650 000	74.5	75	0.0	100	.....	....	..	0.55
83	Feb. 1	" 2	23	13	0.50	1.50	730 000	83.7	60	0.0	100	.....	....	..	0.50
84	" 2	" 2	12	47	0.60	1.86	810 000	93.0	50	0.0	100	.....	....	..	0.50
85	" 2	" 3	10	11	0.60	1.46	790 000	90.7	30	2.0	93	.....	....	..	....

The area of the filter = 0.008719 acre.

Air Wash.—The roughing filters, when clogged, or when not yielding a satisfactory effluent, are cleaned by washing with air and water, first shutting off the raw-water supply and then, if pushed for time, opening the sewer gate and wasting the water upon the surface of the filter down to the level of the wash-water troughs. The filter, in the meantime, being in operation, is allowed to run until the surface of the water on it has dropped about 6 in. below the edges of the wash-water troughs; the effluent gate is then closed, and compressed air is discharged into the underdrains from the air receiver. The compressed air is stored in the receiver at a pressure of approximately 100 lb. per sq. in. at the beginning of the wash, and is discharged through a reducing valve set at about 10 lb. on the filter side of the valve; this, however, does not represent the pressure at which the air is applied during the wash, as the 10 lb. is largely used up in friction through the small air pipes leading to the filters. The air issues into the filter through the orifices in the bottom of the 2-in. galvanized-iron underdrain pipes 1 in. above the floor of the filter. The bottoms of the pipes lying all in the same plane, the air issues from the orifices only as it is forced out by the pressure in the delivery pipe, its tendency to issue by its own buoyancy being overcome by placing the orifices in the bottoms of the pipes; within limits, therefore, it is possible to deliver to the filter any quantity of air desired, and to have it issue through all the orifices in the underdrain pipes with practical uniformity. The application of the air is continued until the contents of the receiver, approximating 1 500 cu. ft. of free air, is discharged, which takes from 4 to 5 min., as the reducing valve is now set, and this corresponds to approximately 1 cu. ft. of free air per square foot of filter surface per minute. Apparently, this is sufficient, for, as far as can be ascertained, there has been no tendency for suspended matters to accumulate in the filter permanently. The objection to the use of a large quantity of air, or to very heavy air washing, is the likelihood of the disturbance of the gravel underdrains to be followed by the loss of some of the filtering material; and the tendency to aid in producing vertical stratification in the filter, in connection with the water-wash, allowing some portions to clog up completely, and increasing the velocity of filtration through the more porous parts to such an extent as to render the filter ineffective. In order to guard against this tendency, the superintendent, from time to time, stirs up by hand the

filtering materials in the roughing filters, using a rake while the wash-water is being applied; this dissipates effectively any tendency of the materials to collect in vertical strata, and maintains the porosity and efficiency of the filter.

Water-Wash.—After the air is turned off the filters are washed with filtered water from the street mains. The water is applied through the underdrains, and the dirty water overflows into the wash-water troughs and passes thence to the river through the sewer. The wash-water is ordinarily applied at a rate equivalent to about three and one-half times the rate of filtration, or between 8 and 9 vertical inches in the filter per minute, the duration of the water-wash being from 5 to 6 min., and the quantity of water used per wash about 10 000 gal. In addition to that used for washing, about 5 000 gal. are wasted from the surface of the filter at each wash when the turbidity is high and the filters have to be washed frequently. It cannot, as yet, be told with accuracy just what percentage of wash-water will be required during the year, but a prediction of the quantity can be made from data secured at the Harrisburg Testing Station, and at the Harrisburg Filter Plant. The number of washes required will depend on the turbidity of the raw water, as this causes the clogging of the filter and necessitates the use of a coagulant.

TABLE 6.—EFFECT OF TURBIDITY OF RAW WATER ON LENGTHS OF RUNS OF ROUGHING FILTERS AT STEELTON.

TURBIDITY OF APPLIED WATER, IN PARTS PER MILLION.		Number of days per year when such turbidities may be expected to prevail. (3)	Average lengths of runs. (4)
Range. (1)	Average. (2)		
0– 50	25	160	6 days.
51– 75	62	65	24 hours.
76– 100	88	35	17 "
101– 150	125	29	12 "
151– 200	175	23	10 "
201– 250	225	13	9 "
251– 500	375	25	7 "
501– 800	650	7	5 "
801–1 200	1 000	5	4 "
1 200+	1 400	3	3 "

Lengths of Runs.—The data thus far secured from the Steelton plant, in addition to information from the Harrisburg Testing Station, indicate that the lengths of runs from the roughing filters, when operat-

ing at a rate of from 75 000 000 to 125 000 000 gal. per acre per day, would be about as shown in Table 6. Column 1 indicates the range of turbidity of the raw water, Column 2 the average turbidity, Column 3 the approximate number of days per year when such turbidities may be expected to prevail, and Column 4 the lengths of runs of the roughing filters.

Using the data in Table 6, 428 washes per year per filter will be required, calling for a total of about 6 700 000 gal. of wash- and waste-water, assuming that the water on the surface of the roughing filters is wasted at each wash. As it takes about 15 min. to wash a filter, the necessary lost time for the 428 washes would be 4½ days per year, and the output of each filter for the 360.5 days of operation, at its full rated capacity, 1 500 000 gal. per day, would be 540 500 000 gal.; the wash- and waste-water, on this basis, would represent 1.2% of the net output of the filter for the year.

The longest run yet experienced was about 24 days, and the shortest 2 hours 11 min. It is believed that with more experience the shortest runs need not be less than about 4 hours.

TABLE 7.—Average Dose of Coagulant.

Turbidity of applied water, in parts per million.		Number of days per year when such turbidities may be expected to prevail.	Average dose of aluminum sulphate, in grains per gallon.	Average dose of lime, in grains per gallon.
Range. (1)	Average. (2)	(3)	(4)	(5)
0– 50	25	160	0.0	0
51– 75	62	65	0.4	0
76– 100	88	35	0.5	0
101– 150	125	20	0.6	0
151– 200	175	23	0.7	0
201– 250	225	13	0.8	0
251– 500	375	25	1.0	0.25
501– 800	650	7	1.4	0.40
801–1 200	1 000	5	1.7	0.60
1 200+	1 400	3	2.1	0.75

Average Dose of Coagulant.—From the data thus far secured, it would appear that the average quantities of coagulant necessary in the operation of the plant would be about as shown in Table 7, from which a calculation indicates that the average dose required through a series of years would be equivalent to 0.37 gr. per gal., or 53 lb. per million gallons of water treated, which, at $20 per ton, gives the average cost for coagulant as 53 cents per million gallons. For the

first year the quantity required will fall far short of this figure, the average dose from September 16th, 1908, to April 18th, 1909, having been but 0.22 gr. per gal.

Effect of Turbidity on Initial Loss of Head.—The initial loss of head, at the commencement of operation of a roughing filter, is not affected either by the turbidity of the raw water or the quantity of aluminum sulphate used, the effects of these two factors being evident only in the rate at which the loss of head increases, and consequently in the lengths of the runs.

Effect of Rate on Quantity Filtered.—The rate of filtration affects only slightly the total quantity of water that may be filtered between washings of the roughing filters; whatever slight effect there may be apparently indicates that somewhat larger quantities can be filtered between washings at high rates than at low rates, but, of course, at the expense of somewhat more frequent washings.

Operation of Slow Filters.

The slow filters are operated on the continuous plan, the rough-filtered water being controlled so as to stand about 2 ft. deep on the surface of the slow filters.

The method of regulating the rate of filtration need not be again described, it being sufficient to state that the yield of the slow filters is responsive to the draft of the municipal high-pressure pumps, up to the limit of the discharging capacity of the regulating devices on the filters. As filtration proceeds, the slow filters gradually clog up, and the loss of head increases; provision is made to allow for a total loss of head of 7 ft. in the slow filters, which is the full depth of the filtering materials, and the superincumbent water.

Daily Records.—The slow filters were put in operation on September 16th, 1908. The daily records of the operation of Filter No. 1 are given in Table 8, from which it will be seen that the accumulation of the loss of head proceeded with comparative slowness during September, October, and November, while during December the occasional increase in turbidity of the raw water began to have its effect on clogging, bringing the total loss of head up to about 2 ft. on the last day of December. In January, 1909, during which there were two periods of bad water, the loss of head increased to about 6 ft., a gain of 4 ft., during the month, and it became necessary to start scraping.

On the morning of January 30th, therefore, the supply to Filter No. 1 was shut off, the water was drained down to the surface of the filter, and filtration was continued for a short time to allow the water to fall a sufficient distance below the sand surface.

TABLE 8.—Record of Operation of Slow Filter No. 1, September 16th, 1908, to February 1st, 1909.

Date. (1)	Time in service: Days. (2)	Hours. (2)	Loss of head, in feet. (3)	Rate of Operation: Gallons daily. (4)	Millions of gallons per acre per day. (5)
Sept. 16	..	..	0.38	680 000	4.7
" 17	1	..	0.36	650 000	4.5
Oct. 1	15	..	0.33	610 000	4.2
" 4	18	..	0.35	600 000	4.1
" 9	23	..	0.45	600 000	4.1
" 10	24	..	0.30	520 000	3.6
" 14	28	..	0.49	510 000	3.5
" 17	31	..	0.35	500 000	3.4
" 23	37	..	0.40	530 000	3.7
" 28	42	..	0.35	630 000	4.3
" 31	45	..	0.30	530 000	3.6
Nov. 4	49	..	0.40	580 000	4.0
" 14	59	..	0.63	500 000	3.4
" 25	70	..	0.61	500 000	3.4
" 30	75	..	0.75	500 000	3.4
Dec. 1	76	..	0.75	500 000	3.4
" 5	80	..	0.85	500 000	3.4
" 8	83	..	1.70	500 000	3.4
" 11	86	..	1.50	680 000	4.7
" 17	92	..	1.20	500 000	3.4
" 23	98	..	1.60	500 000	3.4
" 26	101	..	1.33	500 000	3.4
" 27	102	..	1.95	500 000	3.4
" 31	106	..	2.07	500 000	3.4
Jan. 1	107	..	2.09	500 000	3.4
" 3	109	..	2.20	500 000	3.4
" 4	110	..	2.35	500 000	3.4
" 5	111	..	2.42	500 000	3.4
" 9	115	..	2.92	500 000	3.4
" 12	118	..	3.70	500 000	3.4
" 15	121	..	3.74	500 000	3.4
" 18	124	..	4.23	500 000	3.4
" 22	128	..	4.12	500 000	3.4
" 25	131	..	5.01	500 000	3.4
" 27	133	..	5.35	500 000	3.4
" 28	134	..	5.99	500 000	3.4
" 29	135	..	5.75	500 000	3.4
" 30	135	7	6.01	500 000	3.4

Filter put in service September 16th, 1908.
Filter scraped January 30th, 1909.
Sand removed, washed, and stored, 18 cu. yd. = 124 cu. yd. per acre.
Quantity filtered during run = 71 000 000 gal.
Millions of gallons per acre filtered between scrapings = 491.
Average rate of filtration = 3 623 000 gal. per acre daily.
Net area of sand surface = 0.1446 acre.
Wasted from surface of filter for scraping .....104 500 gal.
Water drained out, and water used for refilling from below...156 800

261 300 = 0.36% of filtrate.

Scraping.—Filter No. 1 was scraped by the superintendent of the plant and a force of five men, the actual time occupied in piling up

the scrapings ready for transportation to the washer being 3 hours, or at the rate of about 7 sq. ft. of filter scraped per man per minute. The scraping on this filter was rather deep, averaging nearly an inch, and being less than ¾ in. in very few places. A portion of the extra scraping was made necessary by the accidental dropping of one of the manhole covers into the filter early in January. This caused a great disturbance of the sand, over an area of perhaps 100 sq. ft., as the cover turned when it struck the water and penetrated the sand bed on its edge, the lower corner having reached almost to the gravel underdrains. The cover was taken out, and all the discolored sand which had been driven down into the bed from the *schmutzdecke* was dug out and sent to the washer.

Sand Transportation and Washing.—The ejector and washing plant worked satisfactorily as to the handling of the sand, but, as it had not been in use before, the men did not attain a high degree of efficiency in its operation. It is believed that greater familiarity with its use, and the proper regulation of the relative force of the different jets, will permit of handling the sand much more rapidly and efficiently than was done. The total quantity of sand removed from Filter No. 1 was about 18 cu. yd., or 124 cu. yd. per acre of filter surface, to transport which to the sand washer, wash it, and return it to the sand troughs on top of the filters occupied about 8 hours, one man attending to the sand washer, one to the hose returning the sand to the sand troughs, one raking the scraped surface of the filter, and an average of three shoveling the dirty sand to the portable ejector.

After the sand for one filter had all been washed, the portable ejector was taken apart and an irregularly-shaped stone, which had carelessly been left in one of the water mains, was found wedged in the nozzle. This had caused the deflection of the jet of water so that the throat on the discharge side had been cut deeply. After the throat had been replaced with a new one, the original capacity of the ejector was restored, and the scraping and transporting of sand to the washer from the other filters proceeded with more expedition.

Ejector Nozzles.—The nozzle in the portable ejector throws a ⅝-in. jet of water into a throat with a 1½-in opening reduced to 1 in. in diameter 1½ in. from the front of the throat and increasing to 1⅜ in. in diameter at the back end of the throat 7 in. from the face; the nozzle and throat stand 1½ in. apart. The nozzle reduces from 2 in. in

diameter to the $\frac{5}{8}$-in. jet in a length of about 6 in. The nozzles and throats are of chilled cast iron.

The nozzles for the sand washer have a $\frac{19}{32}$-in. jet, and the throats reduce from $1\frac{1}{2}$ in. to $1\frac{1}{8}$ in. in diameter in $1\frac{1}{2}$ in., increasing to $1\frac{3}{4}$ in. in $5\frac{1}{2}$ in. The portable ejector and the washing hoppers were supplied by the Norwood Engineering Company, from designs by Allen Hazen, M. Am. Soc. C. E., originally worked out for the filter plant at Washington, D. C., and containing recent improvements suggested by Mr. Hazen. The washing plant consists of only two hoppers, and practically all the washing is done in the first one, the sand as it is thrown to the second hopper from the first being sufficiently clean to be used in the filter. The wash-water overflows the second hopper with very little turbidity.

The lift from the portable ejector to the top of the pipe discharging into the washing hopper is about 15 ft.; from the second washing hopper to the sand troughs there is a drop of about 6 ft., so that comparatively little pressure is required to transport the sand from the sand washer back to the sand bins.

The washing hoppers are provided with auxiliary jets at the bottom to compensate for the quantity of dirty water actually picked up by the jet and forced through to the succeeding hopper, the auxiliary jet also serving the purpose of stirring up the sand at the bottom of the hopper so as to facilitate its transportation by the water jet. The dirty water overflowing the washing hoppers passes through a reinforced concrete box in order to catch such sand as may escape with the wash-water; the dirty water escapes from this box to the sewer by an overflow.

The water which transports the washed sand to the sand troughs overflows a weir at one end of each sand box and escapes to the sewer.

Refilling after Scraping.—After the dirty sand is all thrown out of the filter and the surface of the clean sand has been raked over to remove footprints and give a smooth even surface, filtered water is admitted through the underdrains until the surface of the filter sand is covered about 2 in. in depth; the raw water is then admitted, gently at first, and the filter is refilled to its proper operating depth with the rough-filtered water.

Resumption of Filtration.—In starting this filter in operation after scraping, the effluent control valve was regulated so that the filter would

deliver, according to its rate-gauge, 800 000 gal. a day, it being necessary to choke down the discharge of the clean filter to a proper rate because the two other filters were very much clogged up and could not yield their proportional part of the water without a loss of head of 5 or 6 ft.; with such a loss of head the clean filter would at once start off with its maximum allowable rate, which was deemed inadvisable, the filter being so new, and having been started in operation so late in the season.

Lengths of Runs.—Filter No. 2 was scraped on February 2d and was put back in service on February 3d, and Filter No. 3 on February 4th, 1909. The lengths of runs of these three filters, therefore, were 135.5, 137.5 and 140.5 days, respectively. Filter No. 1 delivered during its run 71 000 000 gal. at an average rate of 3 623 000 gal. per acre daily, the total yield corresponding to 491 000 000 gal. per acre between scrapings. The figures for the other two filters were a trifle larger.

The total quantity of water wasted from the surface of the filter prior to scraping, and the quantity used for refilling the filter from below, before starting in operation again, corresponded to 0.14 of 1% of the quantity of water filtered.

The three slow filters have now (May 10th, 1909) been in service for 14 weeks since the last (and only) scraping, and the loss of head is but 0.7 ft.

Efficiency of the Plant.

The efficiency of the entire plant, in the removal of turbidity and bacteria, is exhibited in the daily records given in Table 9, which date from November 1st, 1908, for the reason that no bacterial analyses were made prior to November 5th.

The efficiency in the removal of turbidity has been 100 per cent. During November and December, when the turbidity of the raw water was low and the bacteria were not numerous, the effluent water was satisfactory. The same is true during January, with the exception of the second week when, as has already been explained, the superintendent was deceived by the deterioration of his silica-turbidity standards, and the quantity of coagulant was inadequate to produce a satisfactory effluent. The results subsequent to that date are good. The samples collected on the afternoon of February 5th, when Filter No. 3 was being scraped, represent the bacterial contents of the effluents of Filters Nos. 1 and 2, which had been scraped, No. 2 within two days

TABLE 9.—Daily Efficiency of Plant in Removal of Turbidity and Bacteria.

Date.	Average turbidities, in parts per million:				Bacteria, per cubic centimeter:				Average dose of coagulant, in grains per gallon.	Alkalinity of raw water, in parts per million.
	Raw water.	Rough-filtered water.	Filtered water.	Percentage of removal.	Raw water.	Rough-filtered water.	Filtered water.	Percentage of removal.		
1908.										
Nov. 1...	5	0	0	100	.....	.....	.....	.....	.....	.....
2...	4	0	0	100	.....	.....	.....	.....	.....	.....
3...	3	0	0	100	.....	.....	.....	.....	.....	.....
4...	2	0	0	100	.....	.....	.....	.....	.....	.....
5...	3	0	0	100	.....	.....	.....	.....	.....	.....
6...	3	0	0	100	80	.....	3	96	.....	.....
7...	3	0	0	100	.....	.....	.....	.....	.....	.....
8...	3	0	0	100	.....	.....	.....	.....	.....	.....
9...	2	0	0	100	.....	.....	.....	.....	.....	.....
10...	2	0	0	100	.....	.....	.....	.....	.....	.....
11...	2	0	0	100	.....	.....	.....	.....	.....	.....
12...	2	0	0	100	.....	.....	.....	.....	.....	.....
13...	2	0	0	100	.....	.....	.....	.....	.....	.....
14...	2	0	0	100	.....	.....	.....	.....	.....	.....
15...	1	0	0	100	.....	.....	.....	.....	.....	.....
16...	2	0	0	100	.....	.....	.....	.....	.....	.....
17...	3	1	0	100	.....	.....	.....	.....	.....	.....
18...	3	1	0	100	.....	.....	.....	.....	.....	.....
19...	2	0	0	100	.....	.....	.....	.....	.....	.....
20...	2	0	0	100	119	76	6	95	.....	.....
21...	2	0	0	100	.....	.....	.....	.....	.....	.....
22...	2	0	0	100	.....	.....	.....	.....	.....	.....
23...	2	0	0	100	.....	.....	.....	.....	.....	.....
24...	2	0	0	100	.....	.....	.....	.....	.....	.....
25...	2	0	0	100	.....	.....	.....	.....	.....	.....
26...	1	0	0	100	.....	.....	.....	.....	.....	.....
27...	1	0	0	100	.....	.....	.....	.....	.....	.....
28...	1	0	0	100	214	12	3	99	.....	.....
29...	1	0	0	100	.....	.....	.....	.....	.....	.....
30...	1	0	0	100	.....	.....	.....	.....	.....	.....
Dec. 1...	1	0	0	100	.....	.....	.....	.....	.....	.....
2...	1	0	0	100	.....	.....	.....	.....	.....	.....
3...	1	0	0	100	.....	.....	.....	.....	.....	.....
4...	1	0	0	100	.....	.....	.....	.....	.....	.....
5...	1	0	0	100	.....	.....	.....	.....	.....	.....
6...	1	0	0	100	408	310	52	87.2	.....	.....
7...	1	0	0	100	.....	.....	.....	.....	.....	.....
8...	1	0	0	100	.....	.....	.....	.....	.....	.....
9...	1	0	0	100	.....	.....	.....	.....	.....	.....
10...	1	0	0	100	.....	.....	.....	.....	.....	.....
11...	1	1	0	100	.....	.....	.....	.....	.....	.....
12...	2	1	0	100	220	125	8	96.4	.....	.....
13...	3	1	0	100	.....	.....	.....	.....	.....	.....
14...	4	1	0	100	.....	.....	.....	.....	.....	.....
15...	4	1	0	100	.....	.....	.....	.....	.....	.....
16...	4	1	0	100	.....	.....	.....	.....	.....	.....
17...	4	0	0	100	1 680	680	71	95.9	.....	.....
18...	7	1	0	100	.....	.....	.....	.....	.....	.....
19...	8	1	0	100	.....	.....	.....	.....	.....	.....
20...	4	1	0	100	.....	.....	.....	.....	.....	.....
21...	5	1	0	100	.....	.....	.....	.....	.....	.....
22...	15	4	0	100	4 425	800	58	98.7	.....	.....
23...	18	2	0	100	.....	.....	.....	.....	.....	.....

TABLE 9.—(*Continued.*)

Date.	Average turbidities, in parts per million:				Bacteria, per cubic centimeter:				Average dose of coagulant, in grains per gallon.	Alkalinity of raw water, in parts per million.
	Raw water.	Rough-filtered water.	Filtered water.	Percentage of removal.	Raw water.	Rough-filtered water.	Filtered water.	Percentage of removal.		
1908.										
Dec. 24...	10	3	0	100	.....	.....	.....	.....	.....	.....
25...	10	3	0	100	.....	.....	.....	.....	.....	.....
26...	6	2	0	100	.....	.....	.....	.....	.....	.....
27...	5	1	0	100	.....	.....	.....	.....	.....	.....
28...	4	1	0	100	.....	.....	.....	.....	.....	.....
29...	3	1	0	100	.....	.....	.....	.....	.....	.....
30...	3	1	0	100	.....	.....	.....	.....	.....	.....
31...	2	1	0	100	.....	.....	.....	.....	.....	.....
1909.										
Jan. 1...	4	1	0	100	5 800	1 125	100	98.3	.....	.....
2...	2	1	0	100	.....	.....	.....	.....	.....	.....
3...	3	1	0	100	.....	.....	.....	.....	.....	.....
4...	3	1	0	100	.....	.....	.....	.....	.....	.....
5...	3	1	0	100		.....	.....	.....	.....	.....
6...	7	2	0	100		.....	.....	.....	.....	.....
7...	120	4	0	100	.....	.....	.....	.....	.....	.....
8...	250	10	0	100	.....	.....	.....	.....	.....	.....
9...	450	14	0	100	.....	.....	.....	.....	0.5	.....
10...	400	20	0	100	.....	.....	.....	.....	0.15	.....
11...	425	20	0	100	18 200	6 000	1 105	93.9	0.30	.....
12...	450	7	0	100	.....	.....	.....	.....	0.90	.....
13...	250	12	0	100	.....	.....	.....	.....	0.10	.....
14...	180	12	0	100	.....	.....	.....	.....	0.20	.....
15...	125	4	0	100	32 000	6 200	320	99.0	0.35	.....
16...	75	4	0	100	.....	.....	.....	.....	0.30	.....
17...	60	5	0	100	.....	.....	.....	.....	0.10	0
18...	30	4	0	100	.....	.....	.....	.....	.....	.....
19...	25	4	0	100	.....	.....	.....	.....	.....	.....
20...	20	4	0	100	.....	.....	.....	.....	.....	.....
21...	35	4	0	100	5 975	1 445	115	98.1	.....	.....
22...	15	2	0	100	.....	.....	.....	.....	0.25	.....
23...	10	1	0	100	.....	.....	.....	.....	0.25	.....
24...	20	1	0	100	.....	.....	.....	.....	0.25	18
25...	50	2	0	100	.....	.....	.....	.....	0.25	23
26...	770	1	0	100	.....	.....	.....	.....	1.38	26
27...	600	2	0	100	13 160	1 025	11	99.9	0.78	23
28...	490	1	0	100	.....	.....	.....	.....	1.32	16
29...	325	0	0	100	.....	.....	.....	.....	1.35	22
30...	225	0	0	100	.....	.....	.....	.....	1.00	20
31...	100	0	0	100	.....	.....	.....	.....	0.65	22
Feb. 1...	75	0	0	100	.....	.....	.....	.....	0.55	16
2...	60	0	0	100	.....	.....	.....	.....	0.50	18
3...	20	3	1	95	.....	.....	.....	.....	.....	14
4...	20	2	0	100	.....	.....	.....	.....	0.25	.....
5*...	15	0	0	100	3 025	295	188	94	0.25	.....
6...	70	0	0	100	.....	.....	.....	.....	0.25	14
7...	45	0	0	100	.....	.....	.....	.....	0.40	17
8...	190	2	0	100	.....	.....	.....	.....	0.60	14
9...	170	0	0	100	.....	.....	.....	.....	0.60	12
10...	215	0	0	100	.....	.....	.....	.....	0.60	13
11...	170	0	0	100	.....	.....	.....	.....	0.60	12

*Filter No. 3 out of service for scraping; Filter No. 1 was scraped on January 30th, and No. 2 on February 3d.

TABLE 9.—(*Continued.*)

Date.	Average turbidities, in parts per million: Raw water.	Rough-filtered water.	Filtered water.	Percentage of removal.	Bacteria, per cubic centimeter: Raw water.	Rough-filtered water.	Filtered water.	Percentage of removal.	Average dose of coagulant, in grains per gallon.	Alkalinity of raw water, in parts per million.	Free CO_2 in raw water, in parts per million.	Average dose of lime used, in grains per gallon.
1909.												
Feb. 12..	270	0	0	100	16 000	500	47	99.7	0.65	13	....	0
13..	160	0	0	100	....	....	..	....	0.65	15	....	0
14..	80	0	0	100	....	....	..	....	0.35	16	....	0
15..	50	0	0	100	....	....	..	....	0.33	14	....	0
16..	120	0	0	100	....	....	..	....	0.50	13	....	0
17..	200	0	0	100	....	....	..	....	0.70	10	....	0
18..	310	8	0	100	....	....	..	....	0.80	6	3	0.42
19..	190	0	0	100	4 500	115	25	99.5	1.00	6.5	3.5	0.42
20..	125	2	0	100	....	....	..	....	1.20	7.5	3	0.30
21..	300	0	0	100	....	....	..	....	0.90	9	4	0.30
22..	390	0	0	100	....	....	..	....	0.80	9	....	0
23..	465	2	0	100	....	....	..	....	1.10	11	....	0
24..	380	0	0	100	....	....	..	....	1.00	11.5	....	0
25..	275	0	0	100	....	....	..	....	0.85	11.5	....	0
26..	310	0	0	100	....	....	..	....	0.80	10	....	0
27..	310	0	0	100	6 850	70	10	99.8	0.80	11	....	0
28..	260	0	0	100	....	....	..	....	0.80	11	....	0
Mch. 1..	150	0	0	100	....	....	..	....	0.65	11	....	0
2..	120	0	0	100	....	....	..	....	0.65	8	....	0
3..	100	0	0	100	....	....	..	....	0.50	7	....	0
4..	65	0	0	100	....	....	..	....	0.40	6	....	0
5..	52	1	0	100	....	....	..	....	0.40	6	....	0
6..	47	1	0	100	3 425	750	61	98.2	0.00	8	....	0
7..	45	0	0	100	....	....	..	....	0.00	10	....	0
8..	60	0	0	100	....	....	..	....	0.35	11	....	0
9..	65	0	0	100	....	....	..	....	0.35	12	....	0
10..	50	0	0	100	....	....	..	....	0.35	12	....	0
11..	50	0	0	100	....	....	..	....	0.35	11	....	0
12..	90	0	0	100	....	....	..	....	0.35	11	....	0
13..	120	0	0	100	2 325	85	10	99.6	0.40	14	....	0
14..	115	0	0	100	....	....	..	....	0.40	16	....	0
15..	100	0	0	100	....	....	..	....	0.40	14	....	0
16..	75	0	0	100	....	....	..	....	0.30	14	....	0
17..	47	0	0	100	....	....	..	....	0.30	12	....	0
18..	42	0	0	100	....	....	..	....	0.30	14	....	0
19..	45	0	0	100	....	....	..	....	0.30	14	....	0
20..	40	0	0	100	....	....	..	....	0.30	14	....	0
21..	30	0	0	100	....	....	..	....	0.25	14	....	0
22..	25	0	0	100	....	....	..	....	0.25	16	....	0
23..	22	0	0	100	1 000	55	5	99.5	0.25	16	....	0
24..	20	0	0	100	.. .	....	..	....	0.00	15	....	0
25..	20	0	0	100	....	....	..	....	0.25	15	....	0
26..	25	0	0	100	....	....	..	....	0.25	14	....	0
27..	110	0	0	100	*	220	17	....	0.40	14	....	0
28..	140	0	0	100	....	....	..	....	0.55	14	....	0
29..	120	0	0	100	....	....	..	....	0.57	12	....	0
30..	150	0	0	100	....	....	..	....	0.62	14	....	0
31..	135	0	0	100	....	....	..	....	0.57	14	....	0
Apr. 1..	100	1	0	100	....	....	..	....	0.47	12	....	0
2..	85	0	0	100	....	....	..	....	0.45	14	....	0
3..	72	1	0	100	2 950	56	16	99.5	0.40	16	....	0
4..	60	1	0	100	....	....	..	....	0.32	16	....	0
5..	50	1	0	100	....	....	..	....	0.35	17	....	0
6..	50	1	0	100	....	....	..	....	0.40	19	....	0
7..	50	1	0	100	....	....	..	....	0.40	19	....	0
8..	55	0	0	100	....	....	..	....	0.45	18	....	0
9..	100	0	0	100	....	....	..	....	0.52	15	....	0
10..	95	0	0	100	....	....	..	....	0.48	14	....	0
11..	80	0	0	100	....	....	..	....	0.45	16	....	0
12..	95	0	0	100	....	....	..	....	0.50	15	....	0
13..	80	1	0	100	1 500	38	4	99.7	0.45	14	....	0
14..	42	1	0	100	....	....	..	....	0.35	14	.. .	0
15..	140	1	0	100	....	....	..	....	0.49	13	....	0
16..	345	1	0	100	....	....	..	....	0.80	12	....	0
17..	350	20	0	100	8 600	475	2	99.9	1.25	12	3	0.25
18..	340	0	0	100	....	....	..	....	1.00	11	2	0.25

* Raw water sample not taken.

and No. 1 within 5 days of the date of taking the sample. The individual samples of the effluents of Nos. 1 and 2 showed the following counts:

Filter No. 1, put in service, January 31st.
Sample No. 1..... 185 (collected February 5th)
Sample No. 2..... 170 (" " ")
Average..... 176

Filter No. 2, put in service February 3d.
Sample No. 1..... 200 (collected February 5th)
Sample No. 2..... 185 (" " ")
Average..... 198

A slight improvement shows in the effluent of the older filter.

The efficiency in the removal of *B. Coli* is shown in Table 10.

TABLE 10.—PRESUMPTIVE TESTS FOR PRESENCE OF *B. Coli* IN THE RAW WATER, FILTERED WATER AT THE FILTER PLANT, AND TAP WATER IN STEELTON, PA.

Date.	Raw Water.		Filtered Water.		Tap Water.	
	Number of 1-cu. cm. sowings.	Positive indications.	Number of 1-cu. cm. sowings.	Positive indications.	Number of 1-cu. cm. sowings.	Positive indications.
Nov. 20, 1908.	5	3	5	0	5	0
" 27	5	3	5	0	5	0
Dec. 6	5	3	5	0	5	0
" 12	5	1	5	0	..	..
" 17	5	1	5	0	5	0
" 21	4	1	4	1	4	0
Jan. 1, 1909.	5	1	5	0	5	0
" 11	5	5	5	4	5	5
" 15	5	3	5	1	5	0
" 21	5	4	5	0	5	1
" 27	5	5	5	0	5	0
Feb. 5	4	3	4	0	4	0
" 12	4	4	4	0	4	0
" 19	3	1	3	0	3	0
" 27	5	5	5	1	5	0
Mch. 6	5	2	5	0	5	0
" 13	5	3	5	0	5	0
" 23	5	4	5	0	5	0
" 27	..	..	5	0	5	0
Apr. 3	5	5	5	0	5	0
" 13	4	3	4	0	4	0
" 17	4	4	4	0	4	0

The presence of *B. Coli* in the filtered water on December 21st, January 11th, and January 15th was in part due to lack of, or insufficient use of, coagulant in the preliminary process. The dropping of

a manhole cover into Filter No. 3 on January 1st, however, may have contributed to the deterioration of the effluent during the early part of that month.

Thus far, no chemical analyses have been made of the applied water and final filtrate at the Steelton plant, and no data, therefore, have been secured on the degree of nitrification accomplished, or on the other usual changes expected. It is evident that between periods of turbid water, when no coagulant is being used, the slow sand filters will be penetrated deeply by such suspended matters as may pass through the roughing filters, among which would be bacteria, and in this respect the work required of these filters will differ from that performed by the ordinary slow sand filter. During the past few months no visible suspended matters have been left in the rough-filtered water.

In nearly all plants using aluminum sulphate as a coagulant, trouble, either continuously or intermittently, has been experienced with discoloration of the water, generally only the hot water, by iron rust. This trouble has been more or less serious at Harrisburg, Pa., Watertown, N. Y., Charleston, S. C., Hackensack, N. J., and various other places. Thus far, there has been no trouble of this sort at Steelton, although at times as much as 2 gr. of aluminum sulphate has been used per gallon of water filtered.

Cost of Operation.—The plant has not yet been in operation sufficiently long to obtain accurate data, but an approximation for an average year would indicate an expense, per million gallons, when filtering 3 000 000 gal. of water daily, about as follows:

COST OF PURIFICATION OF STEELTON WATER PER MILLION GALLONS, ON A BASIS OF FILTERING 3 000 000 GAL. PER DAY.

Operation of Roughing Filters.

Labor	$1.55	
Superintendence	0.42	
Supplies and analyses of water	0.30	
Coagulant	0.53	
Power	0.08	
Wash-water	0.13	
Light	0.05	
Coal	0.10	
		$3.16

Operation of Slow Filters.

Scraping, 5 times per year	$0.60	
Transporting and washing sand	0.06	
Re-sanding filters	0.25	
Superintendence	0.42	
		$1.33
Total		$4.49

Interest and sinking fund charges are not included.

Acknowledgments.—The principal credit for the existence of the Steelton filter plant is due to J. V. W. Reynders, M. Am. Soc. C. E., President of Councils of Steelton, Chairman of the Filtration Committee of Councils and Vice-President of the Pennsylvania Steel Company, who, by arranging a well-conducted campaign of education and devoting much thought and time to the cause, prepared the way to enable Councils to see the necessity of making provisions for a purer water supply.

On its completion, the plant was turned over for operation to the Board of Water Commissioners, Mr. George H. Roberts, President, under the general direction of the Superintendent of the Water-Works, Mr. O. P. Baskin. The operation of the filters is in charge of Mr. M. B. Litch, as Filter Superintendent, the writer acting in an advisory capacity as required.

The works were built by contract under the supervision of the writer's engineering force, Mr. Paul Hooker being Principal Assistant during the designing of the works, and Resident Engineer during their construction; to his faithfulness the plant bears permanent witness.

DISCUSSION.

Mr. Hinckley.

H. V. HINCKLEY, M. AM. SOC. C. E. (by letter).—The author states that one of the novel features of this plant is "the small area required —about one-fourth of that ordinarily required for a slow sand filter plant of equal capacity," and mentions on page 136, "the very high rate of operation of the slow filters." On page 192 he gives the operating speed as 3 623 000 gal. per acre per day. The average capacity given by authorities to date for the successful operation of slow sand filters is 3 000 000 gal. per acre per day. It seems, therefore, that, as operated, the Steelton slow filters fail to realize four times the speed of successful operation of slow sand filters elsewhere. Possibly the author's meaning is that the pre-filtered water at Steelton can be put through the slow filters four times as fast as the raw water. It would be instructive if the author would clear up this point. It would also be interesting to know whether there was any reason, other than cost, for the use of coal in place of sand in the roughing filters.

Mr. Copeland.

WILLIAM R. COPELAND, ASSOC. AM. SOC. C. E. (by letter).—There are several features of the Steelton filters which will attract the attention of those in charge of the operation of such plants, one being the fact that the works have been run without the supervision of a resident chemist. It seems remarkable that such a changeable river water can be handled without the constant supervision of a technically trained chemist, and it can only be explained by the fact that Mr. Fuertes, aided by a corps of technically trained men, bacteriologists, chemists, and engineers, made a most thorough study of the water at Harrisburg only a few years ago, and that the filter plant at Harrisburg has enabled him to study still further the conditions which prevail at all seasons of the year.

The history of this situation teaches a most important lesson, namely, the value of thorough preliminary study. If it had not been for the experiments at Harrisburg, it is probable that the Steelton plant would not have been constructed according to the present design, and it is doubtful whether it would have been possible to operate it, at least for some years, on the present economical basis.

Table 4 shows that the plant treats a water having an average turbidity of about 300 parts per million, by using an average of only 1 grain of coagulant per gallon—a quantity hardly more than half as much as that used to treat the turbid Ohio and Mississippi waters when they carry equal amounts of turbidity.

The economical application of coagulant was made possible, and was controlled to a great extent, by the fact that Mr. Fuertes, having at hand the data secured at Harrisburg, prepared the diagram, Fig. 5, showing the proper relation between the turbidity of the river water

Mr. Copeland.

and the number of grains of coagulant required. This diagram presents the rather unexpected feature that less coagulant is required for a given turbidity when the river is rising than when it is falling. The author explains this by the fact that the particles which form the turbidity in a rising flood are coarser than those in a falling flood, and settle out more rapidly; moreover, high numbers of bacteria are often found in water of low turbidity, thus requiring extra coagulant.

It is stated that the river sometimes carries a natural coagulant, and to this fact, in large degree, is attributed the economical use of alum. The writer's experience with the Allegheny, Monongahela, and other waters of Western Pennsylvania which carry mine drainage, is that the first flood run-off sweeps large volumes of the acid mine waters into the rivers, or out of the pools in the tributaries, and into the main streams, thereby adding sulphate of iron from the coal-pits and washeries to the mountain waters charged with lime. As Mr. Fuertes points out, the alkalies precipitate the iron salts in fluffy clots which absorb and precipitate the mud, clay, bacteria, etc., carried by the storm-waters from the hills and fields. So potent are these iron salts as clarifiers that the usually turbid Allegheny has been made as clear as crystal for 40 miles, and the bacteria have been reduced from 5 000 or 6 000 per cu. cm. to 2 per cu. cm. Moreover, the acid mine waters have such a poisonous effect on organic life that they kill thousands of fish at such times.

These conditions are often accompanied by another peculiar feature, known to river men as "clear-water" rises. These rises, of course, are nothing more than the flood-waters clarified by the precipitation of the mine drainage. These clear waters start at a place where some acid water enters, and are marked by clumps of brown hydrate whirling around in spots about the size of a bushel basket.

The acid carried by the water at such times neutralizes the lime carbonates in the rivers to the extent of rendering the water acid. The writer once saw a band of Italian laborers driven out of the Kiskiminitas, where they had been putting up a bridge pier, by the acid mine drainage swept out of the Loyalhanna by a thunderstorm.

As the coal mines are being extended every day, it behooves all engineers who are planning water purification works along streams in mining districts to provide means for adding extra alkalies, if need be, to neutralize the acid mine wastes.

Mr. Fuertes has made such provision at Steelton, and attention is called to the large drain pipes under the lime tanks. Large drains are very necessary wherever lime solutions are to be made, and plenty of blow-offs should also be provided. The lime solutions do not corrode iron, but form crystals of carbonate, which, accompanied by the insoluble sludge, in the course of time, will clog the pipes.

While discussing the subject of coagulants it should be noted that plans have been made to reduce the turbidity of the effluent from the

roughing filters at all times to less than 25 parts per million. Engineers in charge of the operation of filter plants have been learning for years that the secret of securing a good effluent from a slow sand filter lies in a thorough preliminary treatment. The cleaner and purer the raw water, the longer the slow filters will run. Ten years ago it was said that a slow sand filter could handle water having a turbidity of 50 parts per million—that is a fact, but it can do so only for a short time and at rates of filtration of less than 3 000 000 gal. per acre per day. Mr. Copeland.

Inventors have spent thousands of dollars and have tested almost every kind of material, from the filamentous sponge clippings and felts made of asbestos or wood fibers, to coke, gravel, slag, etc., but, for the most part, all have proved failures. The reason is plain. One cannot clean such beds without taking them apart, a process which is laborious, slow, and expensive. Unless this is done the beds become clogged so that the water is forced to rush through in constricted areas, at the same time scouring out part or all of the mud which was deposited in the bed during an earlier part of the run.

Mr. Fuertes has taken this matter into careful consideration, and has built his roughing filters out of particles which have an effective size of only about 1 mm. These grains are so small and light that the currents of wash-water applied will suspend every grain and separate it from its neighbors.

Unless the roughing filter will reduce the turbidity of the applied water to 25 parts per million, it is essential to add a coagulant. Where lime and iron are added, the best results can be obtained by applying them as near together as possible, so that the lime is brought into contact with the iron while the former is in the condition of caustic lime (CaO) and not as the mono- or bi-carbonate of lime.

Where the water is first subjected to chemical treatment, for the purpose of "softening," the lime added will often precipitate magnesium in flocculent masses, and these flocs make most excellent coagulant. In fact, the preliminary treatment at a water-softening plant will often remove from 75 to 80% of the suspended matter and 90% of the bacteria by sedimentation. The slogan of an efficient preliminary treatment, therefore, must be: Reduce the turbidity of the raw water to 25 parts per million and the bacteria to less than 200 per cu. cm.

JAMES E. LITTLE, ESQ.*—This paper contains an unusually clear and complete statement of the conditions existing at Steelton and the methods being followed to secure a pure water supply. As might be expected, the citizens of the borough are justly proud of the good showing the filter plant is making, and feel that they owe no small debt of gratitude to its able designer, who has spared no effort to make this plant a success. Mr. Little.

* Member, Board of Water Commissioners, Steelton, Pa.

Mr. Little. In order to corroborate the author's statements, and as a supplement to the paper, some additional data concerning the operation of the plant during its first year are submitted.

The plant has done far better than was expected in the matter of cost of operation, without any sacrifice in efficiency. No material alterations have been found necessary, which fact is notable when the novelty of the design is considered. At present sufficient laboratory equipment is being purchased to enable the superintendent to make bacteriological and microscopic determinations, after which a daily examination of the water will be made.

A form of graphical report of the operations has been devised, which is made up monthly on thin paper. Blue prints from this report are sent to the various officers of the borough who are interested in the operation of the plant. These, when properly pasted together, form a continuous graphical report which has been found of considerable value in comparing operations at different periods. The daily averages of height of river, turbidity of raw water, quantity of chemical dosing, gallons of water filtered, and gallons of water used in washing prefilters are plotted to convenient scales, and the monthly averages are given in every case. The reports of the bacteriological analyses are placed at the top of each sheet.

The average condition represented by this record, which covers the period from January to September, 1909, inclusive, is as follows:

Height of river above low-water mark, in feet..	3.38
Turbidity of raw water, in parts per million...	87.50
Chemical dose, in grains of alum per gallon...	0.237
Water filtered daily, in gallons.....	2 092 000
Water used daily in washing preliminary filters, in gallons.....	38 960

The latter figure indicates that the length of the average run of each filter during this period was approximately 24 hours, as 13 560 gal. are required for each washing.

As stated by Mr. Fuertes, no alum was used until January 9th. From this date until May 15th the dosing was continued in varying proportions, except on nine days. From May 15th to the end of September no alum has been used, except on six days in June. The average dose of alum for a full year's operation was 0.157 grain per gal. This is considerably less than the quantity noted earlier in the year and quoted by the designer. It is expected, however, that further reductions will be made in this particular when more frequent analyses of water are possible.

The efficiency of operation—represented by the percentage of removal of bacteria—has been averaged for each month with the following result:

Mr. Little.

	Percentage removed by preliminary filters.	Total percentage removed.
January	82.38	99.015
February	96.73	99.74
March	89.36	98.98
April	93.14	98.90
May	90.42	99.67
June	91.81	96.49
July	95.82	97.91
August	97.72	97.86
September	90.42	96.78
Averages	91.98	98.37

These figures are not as reliable as they might have been, as analyses were made only once a week, but they give a good idea of the results obtained. After all, a satisfactory report from the borough health officer gives the best idea of the filtration efficiency. During the first year of operation not a single case of typhoid fever was reported, which is an unusual record for Steelton. Unfortunately, during the past few days, some cases have developed, but most of these are readily traceable to outside causes. A Labor Day picnic on a nearby island, when raw river water was used for drinking purposes, is undoubtedly responsible for four cases.

During certain periods of the year, when flood conditions obtain, the character of the water has to be watched very carefully. A notable instance of the suddenness of changes is recorded for January 26th and 27th, when the turbidity increased from 25 to 1 500 parts per million in less than 7 hours. At such times it has been found best to regulate the chemical dose so as to produce an effluent from the roughing filters as nearly constant in character as possible, rather than follow the ordinary rules. Two charts, Figs. 7 and 8, have been prepared, showing the details of operation from April 15th to 19th and May 1st to 5th, which illustrate the necessity for this. In each case the increase in turbidity was accompanied by a change in the character of the water, the high turbidity being caused by yellow clay having a coefficient of fineness of less than one. This was not handled with entire success the first time, but a record of the second period shows that there is no serious difficulty in removing the turbidity entirely from such troublesome water in preliminary filters and producing an effluent which will vary but little from zero. In fact, it has been demonstrated that an actual zero effluent can be maintained, if proper care is taken.

Regarding the cost of filtration at Steelton: Unfortunately, the accounts are not kept in such a manner as to permit making a comparison of the details of costs with those estimated by the designer.

Mr. Little.

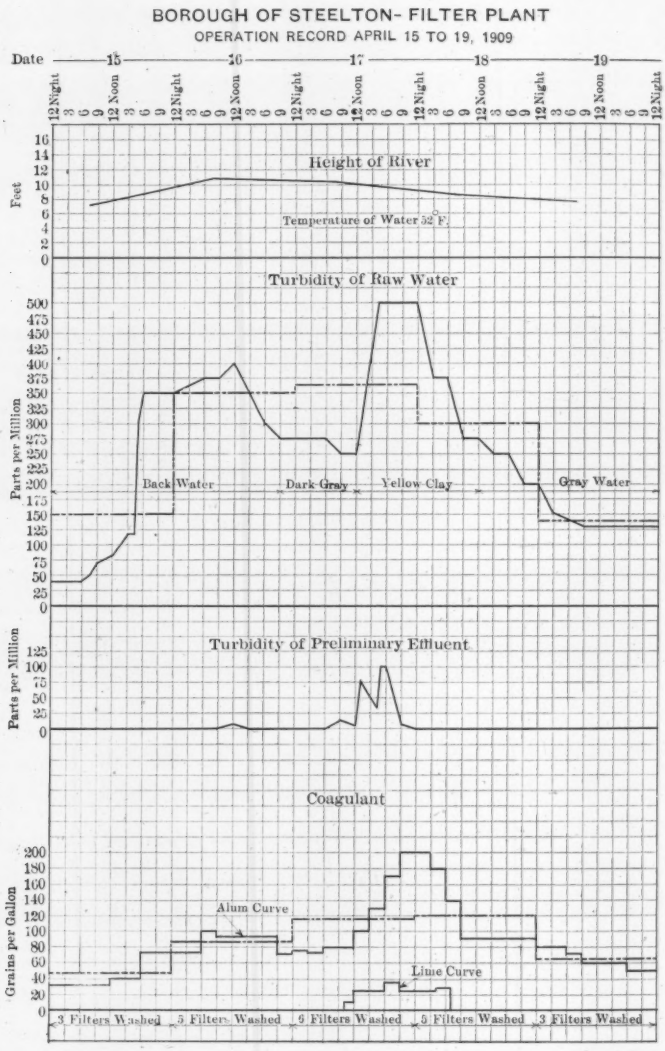


FIG. 7.

Mr. Little.

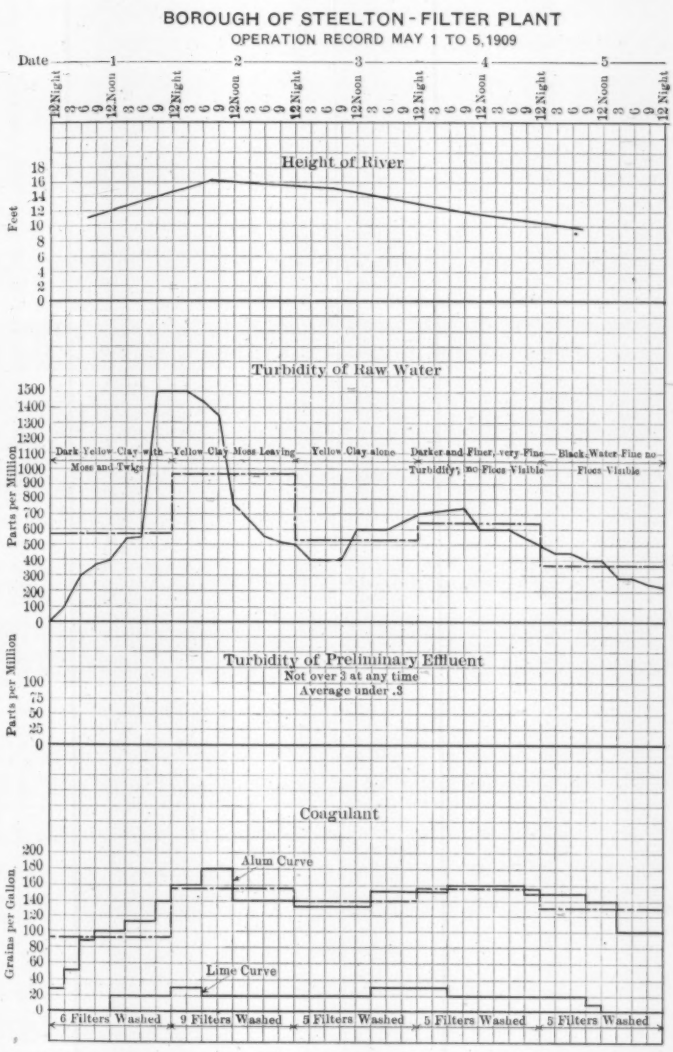


FIG. 8.

Mr. Little. The following statement of totals, however, will give an indication of the results obtained. The cost of operation given includes everything that could be legitimately charged to this account, namely, superintendence and professional service, regular labor and attendance, extra labor, analysis, chemicals, light, heat, power, insurance, repairs, and minor improvements. In order to make a comparison with the designer's estimate, the cost of operation has been estimated on the basis of 3 000 000 gal. daily, by adding the cost of additional chemicals and power required, and by adding proportionally to the charge for scraping the slow sand filters. It is believed that all other charges would be unchanged by an increased output.

	Total operating cost.	Cost per million gallons filtered.	Estimated cost per million gallons, on basis of filtering 3 000 000 gal. per day.
Sept. 12th, 1908, to Feb. 28th, 1909	$1 477.65	$4.505	$2.80
Mar. 1st, to Aug. 31st, 1909.....	1 597.06	4.232	3.027
	$3 074.71	$4.359	$2.914
Value of wash-water used @ $10 per 1 000 000 gal.....	180.88	0.025	0.025
	$3 255.59	$4.384	$2.939

The slow sand filters have been scraped only once during the year. The cost of this scraping was $50.41, which is at the rate of $0.068 per 1 000 000 gal. filtered.

Dr. Rideal. SAMUEL RIDEAL, ESQ. (by letter).—One of the important scientific points clearly brought out by the author, which the writer thinks is novel, is the influence of bacteria on the inorganic suspended matters in the water. Hitherto attention has been confined almost solely to their action on the organic suspended solids and on dissolved matters, though entanglement and sedimentation by fibrous and gelatinous growths have long been known. It is also well shown that the effect of such natural purification is reversed during floods, when the organisms previously deposited are dislodged and carried down, and that, therefore, an artificially controlled system of purification is necessary. The problem at Steelton was one of unusual difficulty, owing to the variability of the water and the conditions, and, consequently, it is all the more creditable that so much success has been attained. The limit of turbidity allowed from the coarse filters, 25 parts per million, is about the same as the permissible limit recommended by the British Royal Commission for sewage effluents, namely, 30 parts per million of suspended solids. This, of course, when drawn with river water into drinking-supply reservoirs, would pass through slow sand filters as it does at Steelton, but generally more slowly.

A fault of the use of aluminium sulphate and lime for coagulating

is the production of sulphate of lime, which correspondingly increases the permanent hardness of the water, and the writer understands from the paper that the amount of hardness and of sulphates in the river at Steelton is already high, also that the water is sometimes acid. He would suggest a trial of sodium aluminate as a coagulant, without lime; this preparation is comparatively cheap, does not increase the sulphate, and, on the other hand, tends to soften the water by removing the lime, iron, acidity, and more or less of the magnesia. It avoids the iron trouble mentioned by the author. Dr. Rideal.

It is difficult to see how the operations can be conducted continuously with safety, without the provision of at least a small laboratory, which, moreover, would simplify the work, with very little expense. Entire dependence on a diagram such as Fig. 5, which is based on experiments made in another locality, seems to savor too much of guesswork, as is practically admitted by the statement that

"The superintendent must watch the effect of his dosing and reduce the quantities if he finds the loss of head increasing on the roughing filters too rapidly, or increase the quantity if the conditions require it," or, "whenever the last bacterial analyses showed 5 000 or more bacteria per cu. cm. in the raw water."

Such a procedure, however, gives no sufficient security as to the bacterial safety of the final filtrates. The bacterial analyses given in Table 5 are made elsewhere at considerable intervals, and vary greatly in results, while only the low quantity of 5 cu. cm. is taken for the *coli* test. The present system cannot be considered a proper method of control, and it is to be hoped, as indicated in the paper, that it is "provisional." The mechanical details present a number of excellent features.

GAVIN N. HOUSTON, M. AM. SOC. C. E. (by letter).—Concerning the percentage of bacteria removed from filters, one often sees the expression, "95% or 99% efficiency." While this catches the popular eye, it means little or nothing to the engineer unless it is accompanied by a statement of the number of bacteria in the raw water. For instance, the highest number of bacteria found in the supply at Fort Collins, Colo., is from 250 to 300 per cu. cm. In this case a filter of from 70 to 75% bacterial efficiency would produce an effluent with from 75 to 90 bacteria per cu. cm., which is well within the limits of modern practice, and yet when expressed in percentage removed is low. The better method of expressing efficiency is by stating the number of bacteria present in the filtered water. Mr. Houston.

The Germans have set the maximum limit for potable water at 100 per cu. cm. The problem in Colorado at present is not that of bacterial pollution, but rather a matter of turbidity, and the general arrangement of the roughing filters in connection with the open sand filter operated at a somewhat higher rate than usual, would seem particularly well adapted to Western conditions.

Mr. Houston. A rate of filtration of 4 000 000 gal. per acre per day has been mentioned as that used in the Denver filters. As this is slightly more than the rate generally used, it would be of interest to know the bacterial content of the raw water and of the effluent.

Mr. Thomas. D. G. THOMAS, M. AM. SOC. C. E. (by letter).—Apparently, the Steelton filtration plant is developing some new and ingenious devices for the treatment of highly turbid water, and the writer has taken a deep interest in their operation. Accounts of the operations at the plant indicate that the problem in this respect is an unusually difficult one.

It has been stated that the mechanical system was installed at Steelton in preference to a slow sand plant because of the unusually turbid water, the average turbidity in the Susquehanna being about 500 parts per million. This is not high when compared with many other streams the waters of which are treated successfully by the slow sand process.

The Platte River in the neighborhood of Denver frequently reaches a turbidity of from 3 000 to 4 000 parts per million, and during the summer of 1909 a maximum of 9 000 parts per million was reached above the intakes of the Denver Union Water Company. During the summer the average condition of the water is practically the same as that of the Susquehanna, although at times it runs as low as 100 parts per million. The problem of treating this water has been solved by the construction of an unusually long sedimentation reservoir with a storage capacity of 300 000 000 gal., or about 10 days' supply for the slow sand filter plant to which it furnishes water. Because of the large capacity and the form of the reservoir, the water consumes about 10 days in its flow from the inlet to the outlet. The result is that, regardless of the turbidity of the inflow, the outflow from the reservoir, as it is applied to the filter beds, seldom exceeds 20 parts per million.

The turbidity of the filtered water supplied to Denver varies from 0 to 1 part per million, which is considered perfect. An interesting example of the efficiency of sedimentation in clarifying the Platte River water was furnished during the summer of 1909. The river above Lake Cheesman was in flood, and the water came into the lake with a turbidity of more than 4 000 parts per million. After traversing the length of the lake, the turbidity was only 2 parts per million. This is as clear as the filtered water from many filter plants in the United States.

The water in flowing through Lake Cheesman traverses about 7 miles. It is recognized, of course, that the conditions which prevail in Colorado make it possible to construct large reservoirs in mountainous districts and to develop ideal situations for the use of the slow sand process of filtration such as cannot be found in the more congested sections of the East where large reservoir sites are too expensive to

consider. Where they can be had, however, the slow sand process will dispose of highly turbid water as effectively and more economically than the mechanical process. Mr. Thomas.

The efficiency and economy are due largely to natural conditions which do not exist in the mechanical plant because of the frequent washings of the filters. Mechanical filters must be washed every few hours; but, as a rule, the slow sand plant requires cleaning not oftener than once a week, and sometimes not oftener than once a month.

In the slow sand plant the cleaning is effected by removing the top layer of sand to such a depth as it is found to be fouled. In the Denver plant, after scraping, filtered water is turned into the bed from beneath, and any silt and bacteria which have penetrated the sand are washed to the surface. The raw water is then turned on the bed, and the effluent is permitted to waste into the sewer for several hours. The result is that the natural organisms in the water and the fine silt accumulate quickly on the surface of the sand, forming a dense coating on the immediate surface, which effectively prevents the passage of additional bacteria into the sand, and, until the bed becomes too badly fouled for the passage of water, does the bulk of the work of purification. The sand itself, without the aid of the coating of silt and organisms, is not fine enough to filter the water successfully.

In addition to its activities in the direction of the purification of the Platte River water by filtration, the Denver Union Water Company prosecutes a careful inspection of the water-sheds of the river and its tributaries above its intakes. It is empowered to do this by a city ordinance and statute which gives the municipality jurisdiction over the stream for a distance of 75 miles above the city. The Company's inspectors are authorized to destroy any nuisances found on the water-sheds, and wilful contamination of the water is a crime punishable by a heavy penalty. The expense of the inspection, of course, is borne by the Company.

JOSEPH A. SARGENT, ASSOC. M. AM. SOC. C. E. (by letter).—Does the bacteriological efficiency of a filter plant necessarily keep progress with mechanical filtration? Mr. Sargent.

With the progress of filtration, can it be said that the harmful bacteria are efficiently eliminated?

Is it not the inevitable conclusion that a rigid sanitary policing of drainage areas should be maintained in conjunction with the workings of the filtration plant in order to eliminate the original accumulation of harmful bacteria as far as possible before the water goes to the filter plant?

The drainage area of the water-shed of the Metropolitan Water Supply of Boston is now under rigid sanitary inspection, as is also the case with the water-shed of New York's water supply. In New Jersey the sanitary policing of water-sheds has not been enforced as

Mr. Sargent.

rigidly as in New York and Boston, and the waters in the latter cities have been freer from harmful bacteria than those of New Jersey towns.

Dr. Jackson, Directing Chemist of the Department of Water Supply, Gas, and Electricity, of Manhattan and Brooklyn, believes that the combination of mechanical filtration with the assurance of a drainage area kept clean by sanitary methods, will reduce the harmful bacteria to a noteworthy degree. Very muddy water, as black as the flood waters of Western rivers, when freed from silt by mechanical filtration may, when clarified, be free from harmful bacteria. On the other hand, is it not true that comparatively clear water, if coming from an unpoliced, unsanitary district having a large population, may pass through the mechanical filter and after such filtration still not be as safe for drinking purposes as waters filtered from a water-shed which gives up a higher percentage of silt and sediment during flood season, but which has been continuously kept free of harmful bacteria by the unremitting vigilance of the sanitary police?

Mr. Whipple.

GEORGE C. WHIPPLE, M. AM. SOC. C. E. (by letter).—The Steelton filter is a veritable *multum in parvo*. Seldom has so complicated a plant been concentrated on so small an area. The situation, however, was unique and called for an unusual design.

Mr. Fuertes' paper is interesting, not so much for its details as for its frank recognition of two important principles in water purification. The first is, that a roughing filter or scrubber is essentially an agent of sedimentation. Mr. Fuertes has used it, not as a substitute for coagulation, but as a means for utilizing coagulation. He has recognized that a scrubber cannot be depended on to remove the finest colloidal matter, and that during periods when the water contains excessively fine particles coagulation is the only remedy. The second principle on which the plant is designed is, that a sand filter is a simpler and safer mechanism to put into the hands of the inexperienced than a mechanical filter of the ordinary type. Those who have witnessed the failure of mechanical filters through the inattention of operatives must fully appreciate this. The force of the argument is partly lost, however, when it is learned that up to the present time the Steelton filter has been operated by a man of technical training. It is possible that this may account for the excellent results thus far obtained, but it should be noted, also, that the actual rates of operation thus far have been considerably below the rated capacity of the plant.

That the results obtained have been satisfactory, from the standpoint of the water consumer, the analytical records well show. That they have been brought about by this new combination of methods, attests the value of the long series of experiments made at Harrisburg, which gave the designer a working familiarity with the varying conditions of the Susquehanna River, which in some respects are unique.

One of the interesting points referred to briefly in the paper is the

statement that there has been no indication of rusty hot water in the city, whereas in Harrisburg, likewise supplied with filtered water from the Susquehanna River, there has been more or less trouble of this character. Whether the freedom from rusty water in Steelton has been due to the use of less alum, or to the use of lime, or to some other cause, will be an interesting problem for the superintendent of the filter to solve. It is to be hoped that the superintendents of the plants at Harrisburg and Steelton will co-operate in carrying on a series of observations in connection with this important matter. Mr. Whipple.

MORRIS KNOWLES, M. AM. SOC. C. E. (by letter).—This very complete description of a most interesting and unique purification works is of great value to all who are engaged in the design and operation of such plants, and the thanks of the Profession are due to the author for placing on record the considerations which led to the adoption of this peculiar combination. Mr. Knowles.

It was thought that the varying character of the waters of the Susquehanna River demanded the flexible method of treatment adopted for Steelton, and the work seems to have been based on the experiments made at Harrisburg prior to building the municipal works at that place. It is to be noted that there are material differences in these two plants, and one wonders why coagulation with rapid filtration is found to be so successful for the larger community, and also whether the greater number of operations found advantageous at the smaller place would have proven economical, both in construction and operation, at the larger; or whether the plant would have been influenced by some local difference in the river water. This point might well be explained.

The data undoubtedly show a low cost, both for construction and, as far as now operated, for maintenance. This is especially remarkable as the plant is of small size, for small operations usually cost more per unit than large ones. It would be interesting indeed if future records were to show that this low cost of operation can be maintained when the painstaking care of the expert mind may be missing, and when age and lack of care have caused the deterioration of some of the apparatus. It would also be of interest to know whether the high rates planned for, and not yet found necessary because of the ample capacity of the plant compared with the use of water in the town, will be possible of attainment at all times, even when the river water is in its worst condition.

The author does not state the cost of the coal used as a filter medium in the roughing filters, as compared with the sand in the finer filters, but it is to be presumed that the former was chosen because of its lower cost, and it would be of interest to know whether a maintenance charge for replenishment, brought about by the more rapid deterioration and wearing away of the coal, due to washing, has

Mr. Knowles.

been considered. It is to be presumed that data on this point, while not yet existing for the Steelton plant, may be obtained from the earlier experiments at the Harrisburg testing station.

The item of greatest interest to the writer is the varying character of the water in the Susquehanna, as it resembles to some extent the occasional condition of the water in the Allegheny at Pittsburg. In the Allegheny a condition of remarkable clearness, due to the admixture of mine drainage and acid water, and possibly iron salts, has occurred occasionally in the last few years, but has been more prevalent during the past season of dryness. Some phenomena, not yet understood, but possibly explained by the author's theory of the action of the Susquehanna River water, resemble the very peculiar effect on the output of the slow sand filtration works at Pittsburg.

The Allegheny in general has a higher bacterial content than the Susquehanna and frequently has much higher numbers—many hundreds of thousands and even millions. It is probable that the Allegheny turbidity is not as great, as a general rule, and does not come from so many different kinds of soil and detritus, and thus at Pittsburg there are not the peculiar varying conditions of the Susquehanna. At times, however, after a dry spell, a marked amount of rain may wash out some of the pools in the affluents of the Allegheny, many of which contain acid waters. These are even strong enough at times to produce acid re-action, and they frequently give the water a clear green color. The crest of a flood may at times contain high numbers of bacteria, yet when there is an acid condition, the numbers will generally be lower, followed by an increase of numbers after this acidity has passed by. At the time of this acid condition, it seems probable that there are iron salts upon which a delayed re-action takes place, causing a brownish precipitate if sufficient time be given. A nominal period of 30 hours or more of sedimentation is not usually enough to precipitate such material, and it frequently passes on to the filters and is precipitated there, causing a clear and satisfactory effluent, but also increasing the loss of head abnormally and shortening the length of run. On the other hand, if the acid is strong enough, these iron salts still remain in solution, and such acid water passes through the filters. There seems to be a certain clearing action, often decreasing the loss of head, which may have previously increased somewhat rapidly. That this is probably due to a dissolving of some of the organic matter in the filters, perhaps even a partial sterilization of the bacterial film on the sand surface, is shown by the additional fact that the bacterial counts frequently increase after water of this character has been applied. Although this increase is marked in total numbers, the number of sewage organisms does not materially increase.

It is remarkable that so small an amount of coagulant is required in the treatment at Steelton, and with the effect that the effluent, as pro-

duced by the roughing filters, is of such low turbidity that the water may be successfully passed through the sand filters at the rates used. It is probable that the continued presence of iron salts and culm wastes permits the use of small amounts of accompanying coagulant with marked beneficial effects. In Pittsburg, however, it has been found that turbidity of a low amount, caused by finely-divided coagulating material, is more troublesome, by clogging in the final sand filter, than the turbidity of the greater amount due to ordinary silt in the river. Mr. Knowles.

GEORGE W. FULLER, M. AM. SOC. C. E. (by letter).—This paper describes a unique and meritorious design which suggests reflection on quite a number of important points learned recently in the field of water purification. Mr. Fuller.

In the first place, the studies of the character of the river water to be dealt with are most highly commendable. This preliminary work seems to have been done thoroughly, to have developed reasonable explanations of the unusual local phenomena, and to have shown in a number of ways why experience elsewhere is not directly applicable in some details to the proper treatment of the Susquehanna River water.

The plant seems to be efficient in the production of a satisfactory grade of filtered water at all times, notwithstanding fluctuations of a striking nature which occur from time to time in the river water. The design of the plant has been carefully worked out, and many commendable and novel details are to be found.

Speaking on broad lines, perhaps the point most clearly accentuated at the Steelton plant is that of the gain for sand filters, with respect to rate of filtration, which results from the application of an influent substantially free from turbidity. This is well in line with the development set forth in practice at various places within the past decade, notably at Zurich, Switzerland, Philadelphia, Pa., Washington, D. C., Albany, N. Y., etc. Incidentally, it is true also with respect to the benefit secured from adequate preparation of the unfiltered water in the case of mechanical filters.

This general subject of high rates has been investigated for a radically different water, namely, that of the fairly clear Croton supply of New York City, where, as a result of tests at Jerome Park Reservoir, it was recommended in 1907 that sand filters be operated at a rate of 10 000 000 gal. per acre daily.

Water supples for municipalities in various districts throughout the United States have a distinct individuality, and in their purification it is quite necessary to design and operate the works in accordance with such individuality. It is difficult, from one set of experiences, to reason clearly and soundly as to the merits of certain devices and procedures elsewhere when not operating under directly comparable conditions.

Mr. Fuller. At Harrisburg, Pa., a short distance from Steelton, a mechanical filter plant is being operated. This plant is of much larger size and is operated under conditions which make it difficult to compare it directly with the Steelton design for the treatment of the same river water, even if all other conditions, including size, are supposed to be substantially the same.

Mr. Fuertes shows that the use of the roughing filter reduces the coagulant to about one-fifth of that required for a mechanical filter plant. That, of course, is due in part to the fact that, for many days during the year, the coagulant for the roughing filter is not used, and in part to the reduction of the dose to somewhat less than that used with a mechanical filter for the same grade of water. The quality of the filtered water from the two types of filters, of course, could be kept substantially the same, and, with the use of a small amount of lime, the so-called "red water" plague could be obviated.

In the writer's opinion, the most pertinent question which this paper presents for discussion is whether, other things being equal, there is a substantial difference in the total cost, including both construction and operation, between plants of the Harrisburg and of the Steelton types. Since the Steelton plant was designed, the sterilization of water, particularly as an adjunct to filtration, has received a marked impetus, and, in reading Mr. Fuertes' paper, the question presents itself as to what a comparison would show regarding the respective total costs per unit volume of Susquehanna River water treated, of the Steelton plant as built and operated, of the Harrisburg style of plant, and of the roughing filter at Steelton operated to produce a clear water at all times, but with some sterilizing agent, such as hypochlorite of lime, used for the elimination of bacteria, instead of the sand filter.

Mr. Diven. J. M. DIVEN, ASSOC. AM. SOC. C. E. (by letter).—The writer has read this paper with much interest—with increased interest because at one time he had considerable experience in filtering a water on the same water-shed, though much farther up than Steelton. He believes, however, that the conditions were identical, except that there were no mine wastes.

The writer's experience indicates that most filter plants are novel to some extent, as the conditions in any two plants are rarely, if ever, exactly the same; and he believes that there is still room for many other novelties which will improve the quality of the water for domestic use, for filtration seems to be in the preliminary or experimental stage. Any devices which will decrease the amount of coagulant required will certainly be hailed with delight, not only on account of the reduction in expense, but because, reasonably or otherwise, there is a popular prejudice against the use of coagulants, the average man believing that where they are used, filtration is simply a chemical

process. The writer, however, will not now discuss the merits of this belief, but rather the methods by which the author has reduced the quantities of coagulants applied to what seems to be a good average river water. Mr. Diven.

The Steelton results are certainly remarkable and very gratifying; but just why the process is spoken of as "slow sand" the writer is unable to understand, for his conception of mechanical filtration is rather along the line of the mechanical action of the chemicals applied as a coagulant than in reference to the mechanical means of cleansing the filter beds—down-drafts and other devices of that kind. His idea of the word "mechanical," as applied to filtration, is the mechanical action of the coagulant in forming the slime coat and entraining the impurities in the water, making it easier to remove them by filtration. To his mind, this is the only difference between slow sand and rapid or mechanical filtration, for all filtration proper is sand filtration. Even in the more modern and improved slow sand filters, mechanical appliances are fast supplanting the old, cumbersome, and not particularly æsthetic method of hand shoveling and scraping the beds, so that if such mechanical devices were to be the distinguishing mark between mechanical and slow sand filters, all filters would soon be "mechanical." To the writer's mind, the description of the Steelton plant tallies in every respect with that of a mechanical plant, notwithstanding the fact that a much larger filtering area is used than has been the practice with the so-called rapid or mechanical filters. The author speaks of the application of coagulants to prepare the water for filtration, describing in considerable detail the very ingenious appliances for regulating and applying these coagulants.

In the writer's mind, the principal consideration in this plant, as compared with the usual form of rapid or mechanical filtration, would seem to be whether the benefits of the small quantity of coagulant required, owing to the large filter area, would offset the extra cost of handling such filter beds, over that of the usual mechanical process of washing the smaller beds. The first cost of constructing the larger beds should also be considered, for there is no doubt that one system would procure exactly as good results as the other, though the average effect on the popular mind of the low use of coagulants should not be forgotten. Whether the use of coagulants, either in large or small quantities, affects the filtered water injuriously, seems to be an open question, and one which the writer trusts will be taken up and settled at an early date; for he believes it would now be within the bounds of possibility to substitute ozonization, or some other process of preparing water for filtration, for the chemicals used in the process. As far as the potability and healthfulness of the water are concerned, all are pretty thoroughly agreed that the moderate and intelligent use of coagulants produces no injurious effect. Of course, if coagulants

Mr. Diven. are used in excess, for the purpose of clarifying the water, the results will be bad. The mooted question seems to be whether or not water treated by mechanical filtration has an injurious effect on metals. In the writer's experience at Elmira, N. Y., this question was never brought up. There seemed to be no increased trouble in the corrosion of pipes or other plumbing fixtures, or in steam boilers, after filtration, even though the quantity of coagulant required there was at times rather high, owing to excessive turbidity at times of flood. The writer was in charge of this plant for seven or eight years after the filters were put in, and during that time had but one complaint of "red water"; this was traced to a defect in the heating apparatus in an apartment house, and the trouble ceased as soon as the heating apparatus was overhauled. However, in four or five years' experience in Charleston, S. C., there has been constant complaint of red water. Whether this is in any way due to the process of filtration, or whether it is due to the quality of the natural water, he is at present unable to state.

The nature of the raw water in these two cases is very different. At Elmira it is rather hard, being about the usual run of river waters in the Eastern States; during low water, it is quite hard and high in alkalinity, but comparatively clear; during flood stages, the hardness and alkalinity are very greatly reduced while the turbidity and color are very greatly increased. In Charleston there is very little variation in the hardness, alkalinity, or color of the water, turbidity being almost entirely absent; it is decidedly a peaty water. The writer's experience indicates that entirely different treatment should be applied to these two kinds of water, if, in fact, the red water trouble is due in any way to the process of filtration. Thus far, the writer has lacked opportunity and appliances for testing this water in its raw state; and, in any case, it would be rather difficult to make such tests, as the water is high in color (about 150 parts per million) and of a decidedly reddish cast.

All will watch with very great interest for further developments at Steelton with a view to applying in some measure the novelties introduced in that plant. The writer feels very much indebted to Mr. Fuertes for his efforts to improve the filtering process, as well as for the very complete description he gives of the Steelton Plant, the process, and the results obtained.

Mr. Fuertes. JAMES H. FUERTES, M. AM. SOC. C. E. (by letter).—The fact that the Steelton filter plant was designed largely on data obtained during the investigations conducted at Harrisburg has naturally brought the Harrisburg plant into the general discussion, and has suggested the query, particularly by Mr. Knowles and Mr. Fuller, as to why these two types, differing in every essential feature, and yet both signally efficient, should have been chosen for the purification of the Susquehanna River water at the two respective locations.

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A mechanical plant, with coagulation and subsidence, was adopted at Harrisburg, instead of a slow sand plant with roughing filters, largely because of the character of the available sites for the plant. Hargest's Island, where the municipal plant was finally located, and a level tract of land more than a mile above the pumping station on the Harrisburg side of the river, were the only sites where sufficient land could be had for a slow filter plant of economical design. Locating the plant on the island would have necessitated surrounding it with dikes 16 ft. high as a protection from floods in the river; and the extra cost of these dikes for the larger slow filter plant, or the extra cost of the long delivery and return mains to the slow filter plant, if located on the Harrisburg shore, in connection with a somewhat more expensive installation, would have made the total cost of the slow filter plant greater than that of the mechanical plant located on the island, and in excess of the available appropriation. The annual cost of operation, however, with all fixed and operating charges included, would have been nearly $5 000 per year less for the slow sand than for the mechanical type. A plant of the Steelton type would have been operated with less difficulty at Harrisburg than the plant actually built, and would have possessed other advantages well worth having; but, as its construction would have necessitated an additional appropriation, its adoption was impossible, in the face of the fact that a type of plant which would be equally efficient, if properly operated, could be installed for an amount within the appropriation.

The actual annual cost of operating the Steelton plant during its first year, as given by Mr. Little, on a basis of filtering 3 000 000 gal. per day, is $2.94 per million gallons filtered. If to this be added interest and sinking fund charges, at 8% on the cost of the plant, the total cost of the filtered water will be a little less than $9 per million gallons. The average cost of the Harrisburg water, as given in the annual reports of the Water Department, has been about $10 per million gallons filtered, all charges included. If based on the nominal capacity of the plant, this cost would be reduced to about $9, or practically the same as the Steelton plant. This should, in a degree, answer the questions of Mr. Knowles and Mr. Fuller, as to the relative costs of operation of the two types, it being borne in mind that the Harrisburg plant has four times the capacity of the Steelton plant, and, therefore, all things being equal, should be expected to show a lower relative cost of operation. The actual costs of operation of each of these plants, as well as their costs of construction, have agreed closely with the estimated costs prior to their construction.

The second reason for the construction of a slow filter plant at Steelton is that the borough was too small to support a properly equipped laboratory, or maintain a grade of supervision requisite for the proper operation of a mechanical plant on so polluted and variable a water.

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The roughing filters at Steelton are efficient. Using an average dose of only grain of alum per gallon, they have permitted running the slow filters at a rate of about 4 000 000 gal. per acre per day for more than a year, with but one scraping, and have produced an effluent, clear, sparkling, and safe for every use. At the Harrisburg plant, more than six times as much alum has been required, per gallon of water filtered, to produce an effluent equally good. The writer, having planned and built both plants, does not mean to imply by this comparative statement that the Harrisburg plant is relatively less efficient or is less efficiently managed than the Steelton plant; the subject is referred to only in response to the comments of Messrs. Copeland, Knowles, and Fuller.

The peculiar clearing action that takes place at times in the Pittsburg filters, as noted by Mr. Knowles, is no doubt correctly explained in the light of local conditions. This phenomenon, however, is not uncommon in connection with slow filters. While reviewing the daily records of the Tome Institute filters, during the summer of 1906, the writer observed that up to May 11th the filters had been behaving normally, requiring scraping at intervals of from 3 to 6 weeks. In the early part of June, however, when the loss of head had reached almost the allowable limit, it ceased for a few days to increase; then from June 15th until the middle of July it slowly decreased until it was no greater than if the filter had been just freshly scraped. By September 7th the filters were again clogged up, so that they had to be scraped; and during the rest of the year no further irregularities of this nature were observed.

On investigation, the writer found that the filters at Washington, D. C., had behaved in the same manner during practically the same period. It was during that summer, also, that Mr. Whipple found that the exhaustion of the dissolved oxygen in the water of one of the Newark reservoirs, due to the sudden death and decomposition of a flourishing vegetable growth therein, resulted in the smothering of great numbers of fish. Possibly, therefore, during the summer of 1906, in this general locality, atmospheric, thermal, electrical, or other conditions of a nature not recognized, may have affected unfavorably the growth and development of certain forms of microscopic plants, the exhaustion of the oxygen in the water, due to their death and decomposition, inhibiting other growths to a sufficient extent to stop further clogging and even allow the accumulations on the tops of the filters to be slowly washed down deeper into the sand beds.

It is probable, therefore, that the action noted by Mr. Knowles at Pittsburg is not dissimilar in its practical effect to that which took place at Washington, D. C., and at Port Deposit, Md., in 1906, although in Pittsburg the effect is attributed to the destruction of the bacterial life in the filter by the cold water. Certain of the troubles with the

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Pittsburg water, as cited by Mr. Knowles, are caused by conditions quite similar to those found in connection with the Susquehanna River water at Harrisburg and Steelton, accentuated with respect to acidity, but tempered with respect to the clogging value of the suspended matter. A more flexible method of preliminary preparation, to take care of these conditions, would doubtless lengthen the runs of the slow filters and increase the capacity of the plant.

Replying to Mr. Hinckley's remarks to the effect that, apparently, "as operated, the Steelton filters fail to realize four times the speed of successful operation of slow sand filters elsewhere," the speed of operation quoted by Mr. Hinckley, viz., 3 623 000 gal. per acre per day was the actual average rate of operation of Filter No. 1 from September 16th, 1908, to January 30th, 1909; the yield of the slow filters, when operating at their full rated capacity, will be 10 373 000 gal. per acre per day, as stated on page 150. Mr. Hinckley's surmise as to the writer's meaning is not correct; there are times when the raw Susquehanna River water could not be passed through the slow filters (without preparation of some kind) for 5 min., without putting the entire plant out of service, so heavy and sticky is the turbidity at such times. What the writer stated, and intended to make clear, was that with the preparatory treatment in use at Steelton the Susquehanna River water, at all times and under all conditions yet experienced on that stream, can be successfully filtered through slow sand filters at a rate of 10 000 000 gal. per acre of filter surface per day, and that the resulting plant, including all the necessary apparatus, will occupy only about one-fourth the area required for a slow sand filter plant with its necessary settling basins, the filters being built to operate at rates suitable to secure satisfactory purification.

Replying to the inquiries of Messrs. Hinckley and Knowles, as to the reasons for the use of coal instead of sand in the roughing filters: the main considerations were that, being of lesser specific gravity, and being composed of smooth particles, the beds of coal could be washed with smaller quantities of wash-water than heavy sand or gravel beds, thus effecting an economy, not only in the operation of the plant, but in its construction, as the sizes of the wash-water pipes could be thereby reduced. Furthermore, if, in course of time, it should be found that the coal particles wear away by attrition during washing, the coal could be removed and used for fuel. Thus far, after 15 months of operation, there is no sign of its deterioration. There was no material difference in cost between the coal and properly prepared gravel; the contractor's prices, when reduced to comparable bases, were the same for the coal in the rapid filters and the sand in the slow filters.

The red water trouble, briefly touched upon by Dr. Rideal, Mr. Whipple, and Mr. Fuller, is quite widespread, not only in cities using mechanical filters, but in others where no chemicals are used in con-

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nection with the treatment of the water. Its causes and prevention, notwithstanding its prevalence, are not generally understood.

In most instances the trouble is limited to the hot water, although sometimes it extends to the cold water also. At Harrisburg the trouble has never been acute, nor general, nor continuous. Normally, the Harrisburg water, after filtration, shows very little free CO_2; probably not more, and frequently less, than an average of about 6 parts per million. In some waters, and under certain conditions, several times this quantity will not cause trouble. At the Jacob Tome Institute, some years ago, owing to an accident to the original filters, the unfiltered reservoir water was used for several months. During this time the water attacked and pitted the boiler tubes and crown sheets at the central heating plant, as well as the steam mains, pipes in the hot-water heaters in the kitchens, and every portion of the piping system conveying hot water and steam, the hot water being unfit for any uses about the school. At this time the free CO_2 in the cold water was about 30 parts per million. Shortly afterward the new filters were completed and the conditions were altered by the separation of the spring water, which normally carries about 80 parts of free CO_2 per million (and formerly ran into the reservoir), from the surface water impounded in the reservoir. The spring water is aerated twice and mixed with the reservoir water in a mixing basin from which the slow filters take their supply. Usually, as it leaves the plant, the filtered water carries about 12 parts of free CO_2 per million. Immediately after the introduction of the new filtered water the red water trouble and all action on the boiler tubes and plates ceased, and both hot and cold water have since remained perfectly clear and free from iron. In this plant, therefore, twice as much free CO_2 as is found at the Harrisburg plant fails to make trouble. At Steelton there has been no trouble with red water.

These circumstances suggest that there may be some connection between the activity of the free CO_2 and the conditions under which it is formed, or the conditions to which the water is subjected during filtration. Mr. Whipple's suggestion that possibly the absence of the trouble at Steelton may be attributable to the use of less alum, or to the use of lime, is not supported by the evidence at hand, as frequently doses of alum, as large as those which were used at Harrisburg when the trouble was prevalent at that city, have not caused trouble at Steelton. Thus far, only a very small amount of lime has been necessary at Steelton, and no attempt is made to neutralize with lime the free CO_2 in the raw water, unless the alkalinity is so low as to require reinforcement in order to combine properly with the aluminum sulphate. This red water question should receive more attention than has hitherto been given to it by chemists in charge of water purification plants.

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Dr. Rideal's statement that the permanent hardness of the water is increased by the use of aluminum sulphate is true. The reactions following the introduction of aluminum sulphate into a water containing alkalinity change a part of the hardness from the bicarbonate to the sulphate form, that is, from "temporary" to "permanent," without increasing the total hardness. The turning of a part of the "temporary" hardness (which could be removed by boiling) into "permanent" hardness (which could not be thus removed from the water in an equivalent degree) would render the hot water harder than if the change in the character of the hardness had not taken place prior to boiling, and, since soap is used more extensively with hot than with cold water, the practical effect of the reaction, in its relation to soap consumption, would be actually to harden the hot water.

Dr. Rideal's further suggestion of the substitution of sodium aluminate for aluminum sulphate has much merit, and should receive careful consideration in connection with American conditions.

The diagram, Fig. 5, showing the amount of coagulant to be used for different conditions of the river water is not in any sense based on guesswork, nor is the operation of the plant left as much to guesswork as is the ordinary type of mechanical filter. Bacterial analyses, the results of which come too late to be of use except as indicating whether the guess, two days earlier, in dosing with coagulant, was a good or a bad one, are of little value as an aid in the determination of the proper dose of coagulant. They are useful in showing what the plant has done, and invaluable in giving the intelligent and observant operator a clue from which to anticipate, from past experience, the conditions likely to be realized in the immediate future. The roughing filters at Steelton, however, are not intended to produce a high bacterial purification, and, as a rule, the quantity of coagulant used need be sufficient only to produce a relatively clear effluent, not an effluent entirely free from bacteria; the slow filters are depended on to finish the process. For this reason accurate adjustment of the coagulant dosing is less essential than would be the case with ordinary mechanical filters. The curves shown in Fig. 5, as appreciated by Mr. Copeland, represent a critical study of the condition of the Susquehanna River water in the vicinity of Harrisburg and Steelton for more than six years, and are based on the results of more than a thousand bacterial analyses, taken in connection with turbidities, and with doses of coagulant which produced satisfactory results in roughing filters of this type. The curves, however, as explained in the paper, require judgment in their use, which judgment must come from a knowledge of the water gained by practical experience and observation, and probably would not be satisfactory at other plants. The simplicity attending their use, and the definite criteria by which the superintendent can judge whether to adhere to the curves or depart from them, render

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them invaluable in securing satisfactory results as well as economical operation.

The operations at the plant, with respect to economy, would be facilitated, of course, by the installation of a small laboratory for chemical and bacteriological work, whereby more complete records could be secured than by the system heretofore in use. Such facilities are now in course of preparation, although not essential for the safe operation of the plant.

The writer is in general accord with the views expressed by Mr. Sargent, as to the desirability of sanitary policing and inspection of water-sheds, as adjuncts of, but not as substitutes for, purification methods. In the case of the Susquehanna, as with others of our large water-sheds, such policing and inspection would be impossible, and valueless as a preventative of water-borne diseases. On small water-sheds it is practicable to protect the water, in a measure.

Mr. Thomas mentions the efficacy of the subsiding basins at Denver in preparing the water for slow filtration, citing the fact that, the storage being equivalent to about 10 days' supply, the slow filters do not require scraping oftener than weekly or monthly. At Steelton, excluding the cost of the land, a storage reservoir holding 10 days' supply would have cost, alone, three times as much as the entire Steelton filter plant, including the land and the slow filters; and the interest and sinking fund charges on the investment would have been nearly twice the total combined fixed and operating expenses of the Steelton filter plant. Further than this, the slow filters in Steelton, instead of being scraped weekly, or monthly, have been scraped but once in 15 months.

Mr. Whipple has commented on the writer's statement that the action of a roughing filter is essentially like that of a settling basin, with a larger area of bottom; this is true not only of a roughing or coarse-grained filter, but, under some conditions and to a certain extent, of both slow and mechanical filters. To illustrate this, which is important, and throws much light on several hitherto imperfectly understood phenomena, two sets of diagrams are herewith reproduced, one showing the losses of pressure and the other the relative clogging, at different depths, of one of the Harrisburg mechanical filters, on runs on January 9th and June 7th, 1909.

At Harrisburg the temperature of the water has a marked effect on the character of the coagulation of the suspended matters in the water. When the water is warm the coagulated particles are large, flocculent, easily visible, and sticky, and therefore, are largely retained on the surface and in the top layers of the sand bed. When the water is cold, however, the coagulation is imperfect, the particles are very small, and almost without the property of agglomeration into larger masses. At such times, therefore, the particles penetrate the

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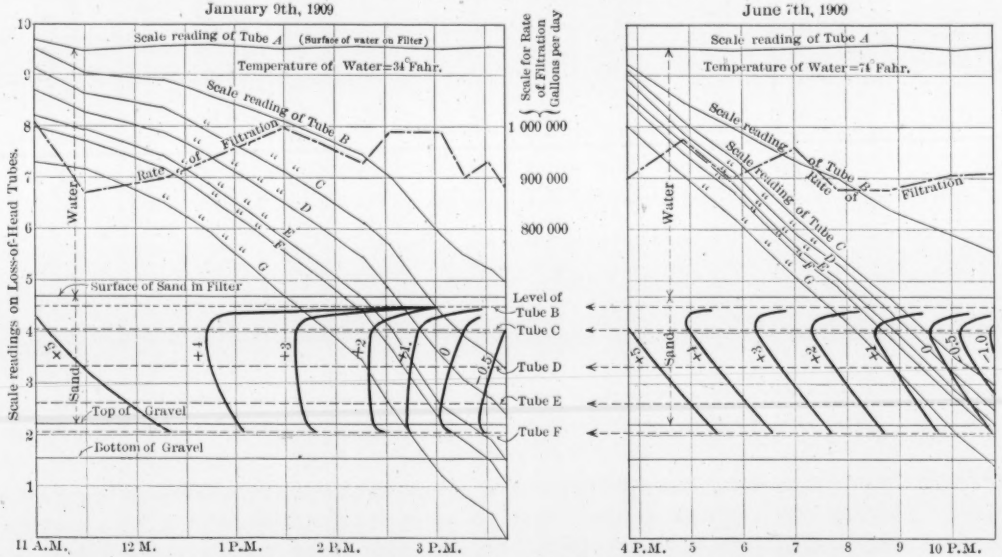
sand bed deeply, gradually clogging the pores, and the filters show increased losses of head at all depths below the surface of the sand, as the length of the run increases. By examining the two diagrams on Fig. 9 it will be seen that for about 1½ hours after the filter was put in operation on January 9th the losses of head at different depths in the sand bed showed a progressive increase throughout the entire run, with apparently no significantly greater clogging at the surface of the filter than at lower elevations. In other words, during this time the particles were not stopped on top of the filters, but passed down deeply into the bed, and were retained therein by subsidence upon the sand grains, by adhesion thereto, and by lodgment in the interstices between them. On June 7th, however, it will be observed that, during the entire run, there was scarcely any increase in the loss of head in the lower 20 in. of the sand bed, practically all the clogging having taken place, from the start, in the top 8 in. In this case the filter acted as a strainer during the entire run, instead of like a settling basin, while in the January run the reverse was the case.

In the diagrams on Fig. 10 this is shown by the relative resistances in the different parts of the filter at different times on the two dates. The greater relative resistances in the lower part of the filter in January than in June show the penetration of suspended matters and their retention, during cold weather, throughout the whole depth of the filter.

Further, by examining the pressures within the filter bed at different times on the two dates, it will be found that when the suspended matters were penetrating the sand bed, in the winter, a condition was reached, about 3 hours after the filter was put in operation, when the clogging was just sufficient, throughout the entire depth of the bed, to require a constant head of 2.5 ft. pressure to force the water through the lower 20 in. of sand at the prevailing rate of filtration. Prior to that time the pressure in the sand bed had been less at the level of Tube *C* than at the bottom of the filter, while subsequent to that time the pressures at the level of Tube *C* was greater than at the bottom of the filter. From the diagram for June 7th, however, it will be seen that at all times during the run the pressures at the level of Tube *C* were less than at the bottom of the sand bed. In other words, during cold weather the filters store up the suspended matters throughout their whole depth, while in warm weather these matters are retained largely on the surface, the lower portion of the sand bed remaining relatively clean. These diagrams are typical of summer and winter conditions; they were not specially selected to illustrate a theory, but are two of several hundred the writer has prepared in making studies of other matters foreign to this discussion.

The diagrams for January 9th represent fairly well the conditions that obtain in the roughing filters at Steelton, as to the penetration

LOSSES OF HEAD AND PRESSURES WITHIN THE SAND BED OF FILTER NO. 2 OF THE HARRISBURG FILTER PLANT DURING THE RUNS ON JANUARY 9TH AND JUNE 7TH, 1909.



NOTE: Heavy black lines show pressures within the sand bed at different times. The line marked 0 shows when the water in the filter was at atmospheric pressure; the line marked + 1 indicates a pressure in excess of atmospheric pressure producible by a head of 1 ft.; the line marked — 0.5 indicates a partial vacuum producible by a head of 0.5 ft. of water

The fine curved lines are the recorded loss-of-head tube readings plotted to the scale on the left at the times corresponding to the scale at the bottom. Fine horizontal lines are drawn at the level of the top of the sand, the bottom of the sand, and the bottom of the filter. Dashed horizontal lines are drawn at the level of the points where different loss-of-head tubes enter the filter. Tube *A* connects with the filter above the sand, and Tube *G* with the outlet pipe below the bottom of the filter.

FIG. 9.

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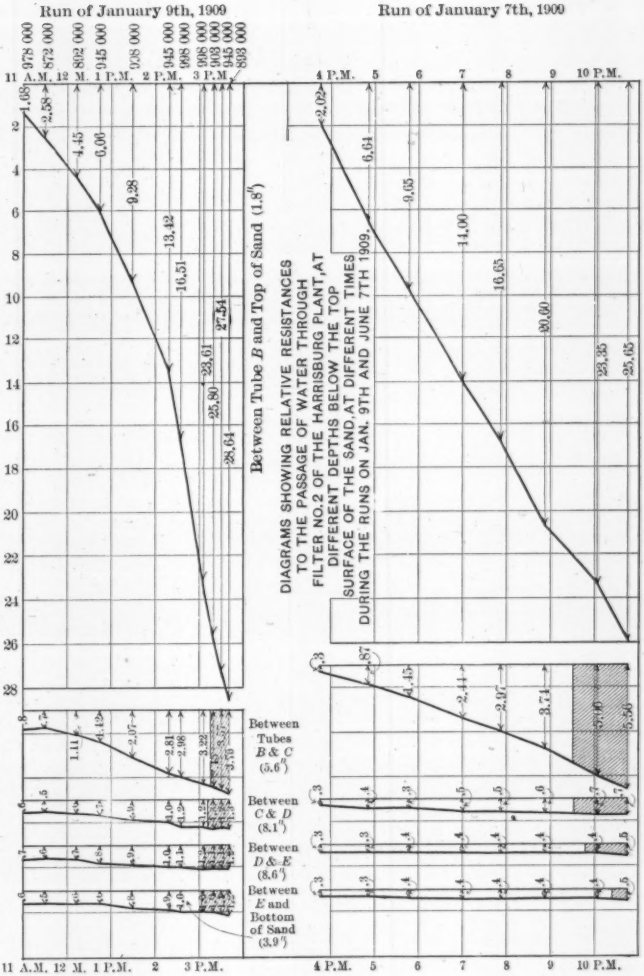

NOTE: The ordinates between each curve and the horizontal line above it represent the head of water, in inches, required to force the water through 1 in. (in depth) of sand, at the levels described at the right, at the times indicated at the bottom, and at the rates of filtration, in gallons per 24 hours, shown at the top.

During the times which are cross-hatched a partial vacuum existed in the filter bed at the levels indicated at the right; the figures in brackets, to the right, are the depths, in inches, between the levels at which the loss-of-head tubes enter the sand.

FIG. 10.

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of suspended matters into the filter bed, but not as to total losses of head, which, in the Steelton filters, is limited by the construction of the plant to about 2.5 ft.

The writer believes that in many cases the irregular and unsatisfactory results from mechanical as well as slow sand filters have in a measure been due to the conditions pointed out above, which could be corrected by using deeper beds, or finer sand, particularly in cold climates. With mechanical filters this phenomenon also has its effect in giving rise to red water, owing to the passage through the filter beds of partly decomposed coagulant, if great and intelligent watchfulness is not exercised over the operation of the filters. To guard against this at Harrisburg the writer directed that during cold weather the filters were to be washed when the loss of head reached from 3 to 5 ft., instead of allowing the full possible head of about 9 ft. to be reached.

The coarseness of the particles forming the filter beds in the Steelton roughing filters precludes the use of high operating heads, and the criteria by which to determine the proper time to wash the filters are the character and appearance of the effluent rather than the total loss of head.

As at present operated, the turbidity of the effluent from the roughing filters is kept as near 0 as practicable, in order to increase the lengths of runs of the slow filters, but it is questionable whether, after all, it is wise, on the score of efficiency in bacterial removal, to allow the roughing filters to do so much of the work. When such a high degree of removal is secured in the preparatory process, the slow filters do not become properly seasoned, and hence, if suddenly charged with an unsatisfactory effluent from the roughing filters, as at the beginning of a freshet, they may fail to deliver a final effluent of satisfactory purity.

Mr. Fuller calls attention to the sterilization of water, as an adjunct to filtration, and suggests that it would be interesting to compare the relative costs of construction and operation, per unit volume of Susquehanna River water treated, of the Steelton plant, as built and operated, of the Harrisburg style of plant, and of the Steelton roughing filter operated to produce a clear effluent, but with some sterilizing agent, such as hypochlorite of lime, used for the elimination of bacteria, instead of the sand filter.

In the writer's judgment, the last method of treatment suggested by Mr. Fuller has merit under proper conditions, but would not be regarded as a safe type for an intermittently and highly polluted stream like the Susquehanna at Steelton and Harrisburg. It must be borne in mind that the beds in the roughing filters are of such coarse-grained materials that the clogging matters in the water penetrate clear through to the bottom and are stored up throughout the whole

depth of the bed. Any irregularity of operation, such as suddenly and abnormally increasing the rate of filtration, or allowing the loss of head to increase beyond a safe limit, which limit is necessarily different for each different kind of water and can only be found out by trial and observation, is likely to cause large quantities of these retained impurities to break through the filter and produce an effluent far more polluted for short periods of time than the original water. This, further, is more apt to occur when the water is relatively clear, and when the numbers of bacteria are relatively low, and, therefore, when little or no hypochlorite of lime is being used, for reasons of economy, than when the water is turbid. For these conditions it will be realized that the Steelton type is infinitely safer than the sterilizing type, the slow sand filters standing as a barrier between the breaking through of the roughing filters and the water consumers. Mr. Fuertes.

The writer has recommended and has recently prepared plans for a plant of the type mentioned by Mr. Fuller. The water to be treated, however, is of a different character from that of the Susquehanna, being derived from an impounding reservoir holding about 200 days' supply, and, though subject to occasional roiliness, not highly and intermittently polluted with sewage. The cost of construction of this plant will be very moderate; and its cost of operation less than that of either the Harrisburg or Steelton types under similar conditions.

In Mr. Diven's conception of the difference between mechanical and slow sand filtration he apparently does not take into consideration the fact that slow sand filters frequently give excellent results with practically no clogging on the surface (or "slime coat"), and that such filters, when properly seasoned, can be raked over on the surface, without interfering with their bacterial efficiency. The greatest difference between mechanical and slow sand filtration is in the rate at which the water is filtered, mechanical filters generally operating at a rate some 30 times faster than is possible ordinarily with slow filters.

Mr. Diven's query with respect to the relative cost of operation of the ordinary mechanical, as compared with the Steelton, type of plant is answered, for the Susquehanna River water, in replying to the inquiries by Messrs. Knowles and Fuller.

The trouble with red water at Charleston has been continuous for a number of years, and at one time was much more serious than it is at present, and probably, as Mr. Diven suggests, the presence of organic matter in solution may complicate the situation and render more difficult the removal of the iron. This condition, as a rule, would call for the use of more lime than is necessary merely to supply alkalinity for combining with the aluminum sulphate; possibly, also, the addition of clay to the water together with the sulphate of alumina and lime, prior to filtration, would increase the effectiveness of the filters in the removal of the iron.

Mr. Fuertes.

The value of properly directed preliminary studies prior to the construction of important water purification plants has been pointed out by Mr. Copeland, Mr. Knowles, Mr. Whipple, Mr. Fuller, and Dr. Rideal, all of whom have been connected with important investigations of this nature, and, therefore, speak authoritatively. One great value of such work, however, has not been brought out in these discussions, and that is that the benefits of such investigations are not limited to those cities for which the work was undertaken. The research work at Lawrence, Mass., Washington, D. C., Louisville, Ky., Cincinnati, Ohio, Pittsburg and Harrisburg, Pa., New Orleans, La., New York, N. Y., Philadelphia, Pa., Columbus, Ohio, and other places on less elaborate scales, has placed at the disposal of smaller communities, not able to pay for such investigations, data and results of estimable value, not only in regard to the proper methods to adopt, but in enabling economies to be effected both in costs of construction and operation. One of the useful results from the Harrisburg experiments, for instance, was the practical demonstration that the type of underdrainage system designed by the writer for the Harrisburg plant was rational, safe, efficient and economical. Its adoption in the Harrisburg plant saved in construction nearly as much as the experimental work cost, barring the laboratory equipment, which was turned over to the municipal plant on its completion.

This type of underdrainage system was used at the Steelton plant, and has also been copied in the plans of several recent and important mechanical plants built by other engineers for cities in the United States and abroad.

Mr. Copeland has also specifically pointed out that the experience gained at the Harrisburg testing station, with the changeable Susquehanna River water, made it possible to put the Steelton plant in operation and keep it going without a break, yielding a satisfactory water in an economical manner from the start. It may not be improper to remark here that much study and close observation of results in the preliminary work were required to perfect the apparently simple arrangement and method of operation in use at Steelton; the problem was nearly abandoned as impracticable on several occasions.

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TRANSACTIONS

Paper No. 1135

THE REDEMPTION OF THE GREAT VALLEY OF CALIFORNIA.

By A. D. Foote, M. Am. Soc. C. E.

With Discussion by Messrs. William W. Harts, H. H. Wadsworth, Erle L. Veuve, Frederick Hall Fowler, George L. Dillman, C. E. Grunsky, A. T. Parsons, D. C. Henny, and A. D. Foote.

The Great Valley of California, as Nature built it, by filling an inland sea with débris from the hills, seemed capable of feeding the world with wheat fifty years ago, when men were reaping from its surface the cargoes for those "swift shuttles of an Empire's loom," the "Gypsies of the Horn."

Fifty years of mishandling natural riches and spurning natural laws have so far injured it that now it may be said, in an economical or engineering sense, the Great Valley is lost to the world. The historic wheat fields have deteriorated until they are no longer of appreciable value to the State. When Mr. Davis, after whom Davisville was named, sat on his housetop and, with a spy-glass, watched his plow teams crawl over his vast wheat fields, he expected a harvest of 40 bushels to the acre. A member of Balfour, Guthrie and Company, the great wheat buyers of the Pacific Coast, recently said that those same fields would now produce from 6 to 10 bushels only, and that of an inferior wheat. Even the mills are obliged to import wheat into California and mix it with the native crop to bring their flour up to grade. Fifty years of crops with no fertilization has made the Valley of almost no economical value.

The two great rivers, which should be 6 to 10-ft. channels, reaching

from Red Bluff to Lake Tulare and from Marysville to the Bay, are shallow sloughs through which a few flat steamboats and barges struggle from one sand-bar to another below Sacramento and Stockton. Fifty years ago they were of immense value to the community as commerce bearers, and, though not large enough to carry the highest floods, they were of vast importance in ameliorating them.

During the winter of 1908-09, it so happened that the first heavy rains fell on the water-sheds of the American and Yuba Rivers. The American rose until the levees on the east side of Sacramento were but 6 in. above the river surface, and the bridges of the Southern Pacific and Western Pacific Railroads went out, though the Sacramento River on the north side of the city was 2 ft. below the high-water mark. The heavy rainfall then shifted to the northern counties, and in a few days the Sacramento River was higher than ever before known, though the American River had gone down. Few realize what good luck it was for Sacramento City that the flood came down the American first. Had the heavy rains fallen in the northern counties first, the crest of the flood would have reached Sacramento about in time to meet the crest of that from the American River. The levees east of the city would have gone out, and the whole American River, swelled by the backwater from the Sacramento, would have swept the city, a mass of wreckage, drifting toward the Bay. A few of the stronger buildings might have stood, but probably the American River channel would now be in front of the Capitol.

The word "luck" is often used to cover ignorance. In this case it covers ignorance of the laws of rainfall. It is not known why the rains came to the American River first, but it is known that it is merely a question of time when the laws of rainfall will swing the rains to the northern counties first and the American afterward, and things will happen as above described.

There can be no reasonable doubt but that the floods in the Great Valley will continue at irregular intervals, and the time will come when the combination of the rain conditions will be such as to destroy, at different times, perhaps, every town and village on its lower levels. No levee system alone can be built that will withstand the great floods. As Mr. Ellery, State Engineer, in his last Report, so well shows, a channel large enough to carry the flood of the past winter between levees is an engineering absurdity. There must be some other method.

The Report of the State Engineer for 1907-08 discusses very ably the apparent antagonism of the different interests in the Great Valley —navigation, flood protection, drainage, irrigation, mining débris— and shows that work done for navigation alone is fatal to flood protection because it contracts the drainage channels in order to give depth at low water, and thus prevents the free passage of the floods. Works for irrigation alone take water needed for navigation. Mining is stopped because the débris fills the drainage channels and spreads over the farm lands. Drainage is blocked by the levee system built for flood protection; and to build levees for flood protection alone is hopeless. The State Engineer also very ably says, in seeking a remedy: "The first requisite is a unification of purpose and harmony of effort among the varied interests involved."

"The solution of the problem for the tiller of the soil essentially solves the whole train of problems," were the words used by Dr. Chamberlain in his paper on "Soil Wastage," read at the second Conservation Conference of the Governors, at Washington.

It is believed that "the solution of the problem for the tiller of the soil" is in harmony with and solves all other problems in the Great Valley, and it is because this problem has been slighted or left out entirely that so little has been accomplished, and so many antagonisms aroused, in the attempts thus far made toward the conservation of its vast natural riches.

During the winter of 1908-09 a correspondent of one of the newspapers mentioned the flooding of 5 000 acres of wheat, but that the owner did not regret it much, as the next crop would be more than doubled by the fertilizer deposited by the water. In this simple statement lies the secret of the redemption of the Valley. It is no exaggeration to state that the floods of last winter carried down enough fertilizing material to produce millions of bushels of wheat, could it have been placed at the disposal of the tiller of the soil.

In Egypt these floods of red water would have been welcomed with feastings and thanksgiving in the mosques, for they would give assurance of bountiful crops. If engineers would study Egypt and follow the teachings of her long experience, in so far as conditions admit, they would be trying no experiment. Willcocks, in his great work, "Egyptian Irrigation," says:

"One cannot study the principles of basin irrigation without admir-

ing the skill and order of the whole operation. However much fault may be found with the unskilful treatment of the new summer irrigation of Lower Egypt, the basin irrigation of Upper Egypt, gradually developed through 5 000 years, commands sincere admiration."

Basin irrigation is dividing the land with dikes into so-called basins and introducing flood-water, usually carrying considerable sediment, from 2 to 6 ft. deep over the entire area, and letting it stand for several weeks until the sediment has settled and the water has soaked into the soil as much as it will. The water is then drained off quickly, and the crop is sown on the mud, often before it is dried sufficiently even to harrow. The areas of these basins depend largely on the slope of the land. In Egypt they vary from a few acres to, in one instance, 40 000, the average being about 5 000 acres. This system of irrigation would be especially beneficial to alkali lands. The experience in Egypt is that where land has deteriorated and shows white efflorescence from perennial irrigation, one or two years of basin flooding restores it to its former state, and in no case has basin irrigation produced alkali lands. So marked is this result that Willcocks intimates that Lower Egypt may yet have to return to basin irrigation, and to wheat raising instead of cotton, tobacco, and sugar, which are much more profitable but require perennial irrigation. Since that time a compromise method of flooding a portion of the land every year with red water has been brought about by raising the Nile at the barrage so as to deliver the red water on the Delta lands, thereby fulfilling Willcocks' famous saying, that the work of the English engineers was to deposit the sediment on the lands and not in the canals. It will be seen, therefore, that in those districts where perennial irrigation has raised the land-water level and brought on the alkali, the basin system will dovetail in and enable the tiller of the soil to wash away his alkali whenever troublesome, and, at the same time, probably lower the land-water level by its quick drainage, as one of the fundamental elements of the basin system is ample drainage channels from every part of the watered area.

No one appears to have realized the great physical and climatic similarity between Egypt and the Great Valley, nor how perfectly the basin system of Egypt adapts itself to and solves the problem in California. Except where perennial irrigation obtains, both countries are limited by the climate to one short-season crop, generally

some food grain, and, as far as can now be judged, this limitation will always hold.

The Egyptians welcome the muddy flood, and gently spread it out over the valley under perfect control, keeping it until it has watered and enriched the land, and then sending it quietly to the sea. Sowing their grain upon the ooze ere the water has barely left it, their crop comes to maturity without other aid. In this way the flood can be mastered and controlled and made to spread peacefully over the floor of the Great Valley, where it will water and enrich the land, and then pass on to the sea, leaving the certainty of full crops behind it. Contrast this with placing dependence on a precarious rainfall to grow an inferior crop on a deteriorated soil, and unsuccessfully fending off the flood in terror lest it destroy the country.

There is one important difference, however, between the Nile and the Great Valley. In the former, the river is expected to fall in time to drain the water from the basins back into it, thus it is both the feeding and draining channel for the basins. Sometimes the expected does not occur, and loss of crops follows. In California the rivers do not usually fall sufficiently to drain the adjoining lands in time for a crop to be planted in the same season, and this may be the reason why the basin system has not been adopted. Other drainage or escape channels must be provided which will drain the basins early in the spring. While these channels will prove expensive, there is ample compensation for them in the fact that the rivers, being high late in the season, can furnish a second irrigation for the wheat in June, which will be of vast benefit to the maturing grain.

It is proposed, therefore, to construct dikes to form basins, as in Egypt, over the entire floor of the Great Valley, comprising some 3 000 000 acres; and to feed these basins, during the winter or flood time, by suitable regulating gates and dams, from the various rivers and creeks entering the Valley; and to regulate and control the feeding of the basins so as to relieve the rivers of flood waters, as much as possible, and hold these flood waters in the basins, or let them move slowly through, that they may deposit the silt and soak the land, and finally drain through escape channels in time for the crops to be sown in the spring. It is proposed to provide escape channels, through the lowest parts of the Valley, of sufficient capacity to drain the basins rapidly, if required, and assist the rivers in times of excessive floods.

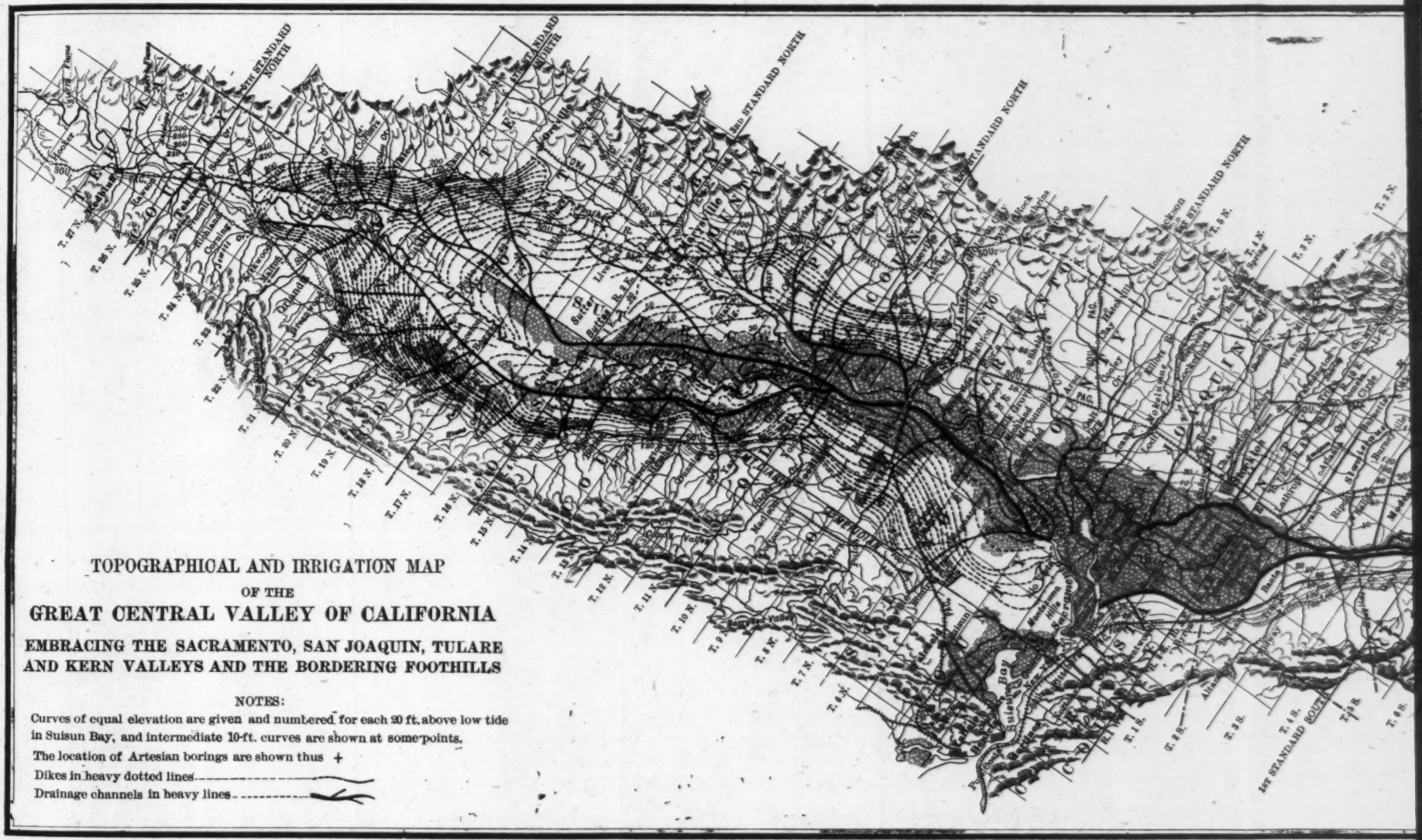
In the Sacramento and San Joaquin Rivers, it is proposed to construct, in connection with the basin-feeding regulators, movable dams with locks, which will give slack-water navigation at low stages of water and offer no obstructions during floods. It is also proposed to construct dams or barriers in the rivers, at the margin of the Valley, which will prevent the movement of mining débris or other material now in the rivers, and to raise these barriers as needed, to prevent the future wash from the hills from flowing into the navigable channels.

It is impossible, of course, in this paper to lay out in accurate detail a plan of basin irrigation and its auxiliary works, for the Great Valley, but it is thought that sufficient may be shown to prove the feasibility of the redemption of the Valley by the means which have been in use for centuries in the Valley of the Nile. The success there proves, before work is begun, that it would not be an experiment.

On the map of the Valley, Plate XXII, which is reduced from the remarkably graphic map of 1887, by William Ham. Hall, M. Am. Soc. C. E., the basin dikes in the Sacramento Valley are indicated in heavy dotted lines as a diagram only, and are not supposed to be accurate. In fact, they are simply interpolated 5-ft. contours. They are not indicated in the San Joaquin Valley, because in that valley there are so many complications with perennial canals and other conditions.

The proper location of the dikes will require not a little care and skill—nearly as much as in locating a railroad line. The surveys and maps now being made by the Reclamation Service are the first necessity. Nearly all the construction can be mechanical. Some dredging or ditching machine of the Aurora type can be especially adapted and built for the service, as there is enough work to be done to pay for special machines. The shape and size of the basins depend chiefly on the slope of the land, for when they are filled there should be a depth of not less than 2 ft. of water anywhere, otherwise the shallower portions will not get their due share of silt.

The dike proposed is 9 ft. high, 8 ft. wide on top, with side slopes of 2 to 1, making a bottom width of 44 ft. This gives $4\frac{1}{2}$ ft. as the average depth of water, with 2 ft. against the upper and 7 ft. against the lower bank. Half the earth would be taken from each side of the embankment, thus making an excavation or ditch which will be of great value in the distribution of the water, both in filling and emptying the basin. An ordinary ditching plow can be used to cut the origi-



TOPOGRAPHICAL AND IRRIGATION MAP

OF THE

GREAT CENTRAL VALLEY OF CALIFORNIA

EMBRACING THE SACRAMENTO, SAN JOAQUIN, TULARE AND KERN VALLEYS AND THE BORDERING FOOTHILLS

NOTES:

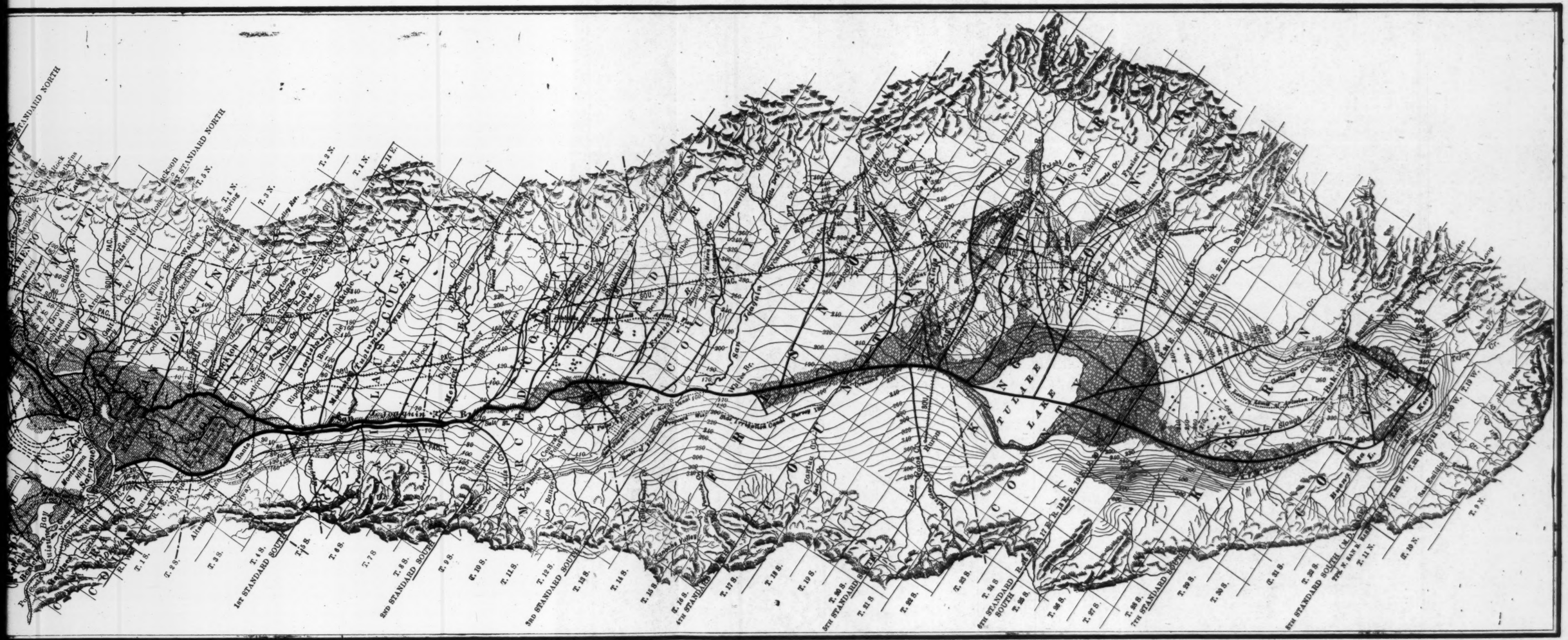
Curves of equal elevation are given and numbered for each 20 ft. above low tide in Suisun Bay, and intermediate 10-ft. curves are shown at some points.

The location of Artesian borings are shown thus +

Dikes in heavy dotted lines

Drainage channels in heavy lines

PLATE XXII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXVI, No. 1135.
[FOOTE ON
REDEMPTION OF GREAT VALLEY OF CALIFORNIA.



nal surface on the center line of the dike, thus making a deep break and a fresh bond with the filling. It is worth while, especially in adobe land, if water can be procured, to settle or puddle the dikes as rapidly as they are built, otherwise they will not hold up much water until after they have stood through at least one rainy season. The dike should be built with the machines, nearly complete, with a trench along the center of the top, and then, at proper intervals alongside the dike, small driven-pipe wells should be put in. With a portable power pump, taken along the line, water can be pumped into the trench and will seep through the entire dike and cause it to settle into a solid mass, care being taken not to water too much nor too fast, thus making mud and causing the bank to slump. There should be drain pipes at intervals along the trench to carry off the heavy winter rains, as, at that time, no water should stand on the dikes. Willow cuttings stuck into the bank will grow rapidly during the first season if kept watered from the trench. These willows are very effective against wave wash, and should be cut every year after the flooding and kept watered through the dry season by pumping into the trench. This may seem to be a trivial matter and a needless expense, but it is of great importance to the safety and permanency of the dikes, and keeps them from being washed by the rains as well as the waves, keeps the earth in place on the slopes, discourages gophers, and tends to a more thorough inspection of the dikes.

It is desirable to feed each basin with water directly from the supply, in order to get all the sediment, and also drain directly to the escape channel, to have it act quickly and thoroughly. In actual practice in Egypt there are many adaptations and complications of this simple proposition, compromising to fit the natural conditions of each locality. There may be a system of basins, each opening to the next, the supply being from the upper to the lower and the draining through the lower from the upper, while the escape channel is by a siphon under the supply channel of another and lower system of basins.

In the Great Valley, the slope of the land is such that most of the basins are very long and narrow, as for instance the 35-ft. contour dike, shown on Plate XXII as starting from near Cache Slough and reaching up the west side of the valley to the Sacramento at Winn's Landing. Both for safety and convenience, this is cut into many basins by cross-dikes, some of them fed from the Sacramento and others from the

side streams, Putah Creek, Cache Creek, etc. When completed, therefore, the basins may be from 10 to 20 miles long and from 1 to 3 miles wide, forming a manageable shape and area.

The feeding regulators, dams, and gates will try the skill of the engineer, especially along the slopes where side streams are to be taken care of. They must be built, not only to feed the basins, but to allow any excess to pass, and they must be built on sand. Much may be learned from the experience of English engineers in India about hydraulic structures on sand foundations. They have done a vast amount of monumental work of this kind, under conditions far more difficult than any in the Great Valley. They have put their head-works and anicuts, as they call them, across sandy-bottomed streams, and these have withstood floods of more than twice the volume ever given by the Sacramento and San Joaquin Rivers combined.

Recent practice in the use of cement grout in sand and gravel foundations will enable engineers to accomplish results in these sandy river bottoms at far less cost than the Indian structures of stone and brick; and the great factories near Vallejo will furnish this most important article, cement, at a price which will allow of its liberal use. Few people know and few engineers seem to realize the value of cement grout for sand and gravel foundations. The cantilever bridge across the Niagara Gorge is built on the loose slide or talus which fell from the bluffs. A place on the slope was leveled off and cement grout was poured in among the loose stones until the interstices were filled; this, on hardening, gave a solid mass on which to build. A cement conglomerate it might be called. The great barrage at the head of the Delta of the Nile, built on Nile mud, was nearly useless for many years until Scott-Moncrieff strengthened the foundations with broken stone filled with cement grout.

The sand and gravel beds of the streams entering the Great Valley are perfect material for forming this cement conglomerate if cement grout is forced into it. The operation is quite simple and rapid. Pipe is driven into the river bed, if possible by the water-jet method, to the depth required for the foundation, and a force pump drives the grout into the interstices until they are full of cement which on hardening forms the solid concrete. The grout may be forced in within a radius of from 3 to 6 ft. from the pipe, depending on the porosity of the bed material. In this manner, broad and deep foundations may be con-

structed on which to build movable dams and regulators, on nearly every stream entering the Valley, at a cost of less than half the usual estimates for such work. Near the center of the Valley, where clay or adobe is encountered, this method cannot be used.

One or two of these structures for regulation will be required on each of the smaller streams, and several on some of the larger ones. Each will have to be designed to fit its own peculiar conditions, after those conditions have been determined by careful detailed surveys. In the navigable streams it is proposed that the feed-regulating structures shall include a movable dam and navigation lock for slack-water navigation. These are necessary in order to turn all the flowing water into the basins, if required, during a dry season, or to open out and afford

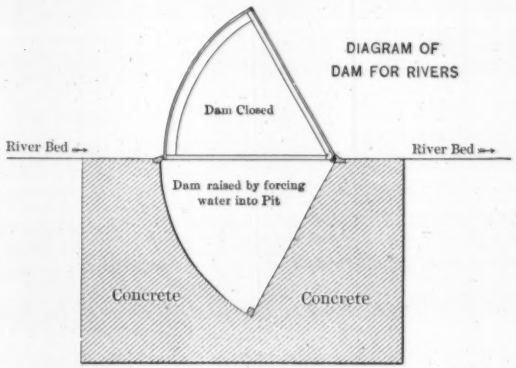


FIG. 1.

no obstruction during a heavy flood season, and also to allow all the water to be used for perennial irrigation after the snow flood in June. For these movable dams and also for the feeder gates it is proposed to use a construction similar to the sector dam or gate designed by Mr. E. R. Cooley and used by Isham Randolph, M. Am. Soc. C. E., at the locks and power plant of the Chicago Drainage Canal. As there is no ice to interfere with them, the only apparent objection to them is removed. If worked by hydraulic power (obtained by pumps, in this case), as at Lockport, it would appear feasible to have a single gate for the entire width of the river, or perhaps two with one pier between them, which, when down, would give a very free waterway for floods. It might be preferable to have the feeder gates in comparatively

short sections, and arrange them with the axis on piers and the gate to be raised out of the water by suitable gear. A counterweight might be added by extending the radials of the structure, as was done by A. J. Wiley, M. Am. Soc. C. E., for canal gates, which makes the manipulation very easy. Figs. 1 and 2 indicate the design of the gates in a general way.

The proposed escape channels are indicated by heavy lines on the map. They are drawn very roughly and tentatively, as they are of the first importance and their location and size can only be determined by hard and skilful work. In the Sacramento Valley, it is proposed to use the Cache Slough, widened to the proper dimensions. With a bottom grade beginning at mean low water, a wide channel should be

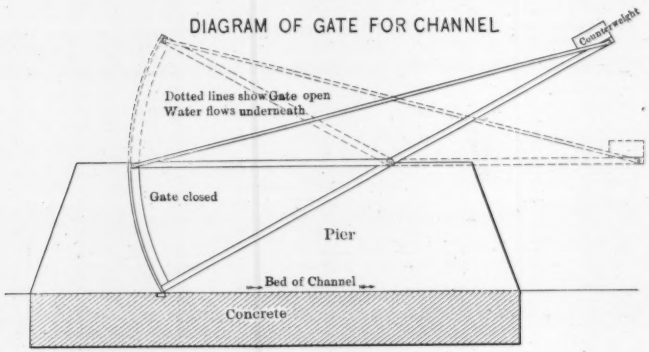


FIG. 2.

carried up the lowest part of the Yolo Basin, with a grade, if possible, which will give from 4 to 6 ft. of natural banks. If feasible, it would be better to use the excavated material for filling or leveling up low places, and have no spoil banks, the design being to use the whole area between the 10-ft. contour dikes as an escape channel when desired, and to use the excavated channel permanently for ordinary drainage and for carrying the excess water of Cache and the other west-side creeks. There would always be a very considerable seepage gathered by the branch channels which would have to be taken away by this main channel.

Above Sacramento a branch channel must be carried under and across the river to drain the American Basin, and should extend north-

ward toward Marysville. There seems to be enough fall to allow of an appreciable head for the siphon under the river, though even then it will need a large cross-sectional area and will be an expensive undertaking. Another branch escape channel will have to be carried under the Sacramento, to take the water from the Sutter and Butte Basins. Adopting for the river the cut-off proposed by the Commission of Engineers, the siphon could be placed in that, while building, and the old channel of the river could be used and extended northward through the Colusa Basin.

The present channels on the west side of the valley, and those of the smaller streams on the east side, can be used both as escape and feeder channels, as their flood season will be over in time to receive the escape waters in the same channel.

The siphons under the Sacramento may seem to be serious obstacles to the system, but on due consideration the objections to them may be reduced to the simple one of cost. There is nothing novel about them except their size, and, if considered as structures only, there is no novelty in their size. The Solani or Nadrai Aqueducts, which pass the Ganges Canal over streams, are far larger and more expensive. Many smaller siphons are in use in Egypt. There are three large siphons in the Cavour Canal, Italy, and as for permanence there is one in Spain, built of masonry by the Moors, which has 300 ft. head and has been in continuous service for more than 600 years, without accident as far as known. If the location should prove to be in sand, the grouting method would reduce the cost materially, and make better foundations, as water would not seep along the grouted portion, and if the upper part were built of concrete the cost would be very low in comparison with the brick and masonry constructions in India. If it is necessary to build these siphons to drain from the basins an average depth of 5 ft. of water in 30 days, it will require a capacity of about 30 000 cu. ft. per sec. for the American Basin and 50 000 cu. ft. per sec. for the Sutter. Assuming that there will be sufficient slope to give a head on the siphon which will force the water through at a velocity of 8 ft. per sec., the siphon for the Sutter Basin would have thirty barrels or archways, each 20 by 10 ft., and eighteen would be required for the American, or the former structure would be about 700 ft. wide and as long as the distance, outside to outside, of the river levees, and the latter would be about 425 ft. wide and of similar length. It is probable

that these siphons and the feeding and regulating dams and locks would be located as one structure in the river, which would reduce the cost appreciably. In any event, it is evident that the cost of the siphons would not be prohibitive, and would be cheaper than any competent pumping plant, even for drainage purposes, if the cost of operation be considered. It should be added that the rectification of the Sacramento River below Cache Slough, as recommended by the Commission of Engineers, is a very necessary part of the escape channel system.

The escape channels for the San Joaquin Valley are also indicated in heavy lines on the map, and have been drawn with considerable hesitation, as there is much less information on record concerning the physical conditions in this valley than in the Sacramento. It is not probable, however, that the San Joaquin River, from the Merced down, can be depended on to subside soon enough to carry off the escape waters from the basins. It is proposed, therefore, to carry an independent channel up on the west side, as shown on the map, to intercept the San Joaquin just above the Merced, and thence, following and enlarging the river, to its bend to the eastward. Thence, it would follow the lowest part of the valley, with a slope sufficient to drain Tulare Lake completely. This would necessitate a long cut from about the 190 contour to the center of the lake, which would be expensive, but it would reclaim the whole area of the lake and its swampy margins. It is probable also that the channel would be continued through the lake and slough to Buena Vista Lake. The San Joaquin River below the Merced will need to be rectified and provided with dams and locks for slack-water navigation, and connected with the upper channel so as to make a navigable channel from the Bay to Lake Tulare. Branch escape channels for draining the basins will be needed, somewhat as indicated on the map.

In the matter of barriers for holding back the mining débris and other wash from the mountains, it may be thought that the State's attempts in the Yuba tend to discourage any more work in that direction. The work done, however, should be remembered rather as a lesson well learned, to be used to future advantage. It appears from the reports that in each instance of the breaching of these barriers it was caused by undermining. That is, the water, after passing over the barrier, dug a hole on the lower side of it and worked back underneath until the barrier fell from want of support. A hint may be

taken from the English engineers in India; for a wall may be carried down into the river bed on the lower side of the barrier far enough to prevent this undermining. This would remedy the immediate and local danger to the barriers, but there is a general action going on which must be remedied also. It is so well known as to need no comment, that rivers if left to themselves are eternally flattening their grade. That is, they are moving whatever forms their bottoms along down stream in the usual course of Nature, and the steeper the grade the greater the action. If, therefore, some extraordinary cause fills up a river and makes its bed much steeper, this cutting action will become rapid, and if, in addition to this, at a certain point in the river, all material coming from above be stopped, and held, then the cutting action below this point will be much more rapid, because, as the material is moved along, there is no other from above to take its place.

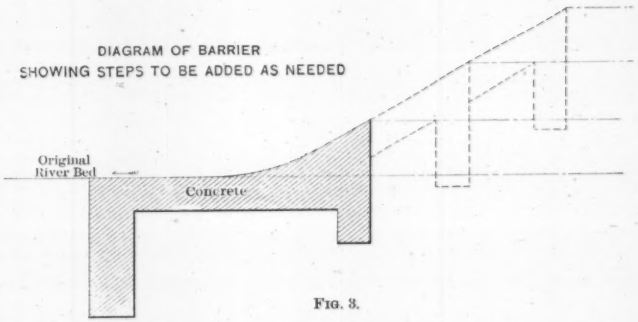


FIG. 3.

If barriers are placed in the Feather and Yuba Rivers up near the hills only, the movement of the vast accumulation of débris below the barriers will simply be accelerated, and they will pass downward into the Sacramento and also soon undermine the barriers. As this material should not be moved into the Sacramento, and as it is desirable to have barriers in order to prevent any more débris coming from the hills, they must be erected at intervals all the way from the Sacramento to the upper barriers; this will break the river grade into steps, with reaches between so nearly flat as to prevent the movement of the coarse débris forming their bottoms. This would lessen the velocity of the current and require higher levees to hold the floods. The excess of the floods must be turned into the basins and pass off through the escape channels.

In building these barriers, shown in Fig. 3, it is proposed to use the cement grouting method for a large portion of the structure. The upper barrier may be added to, as needed, for generations to come, without endangering its stability, and, if the surface becomes worn appreciably, it will be neither difficult nor costly to spread a few feet of concrete over it, during the dry season, and this, if kept moist until it has set, will be as good as the original surface.

As soon as the side streams entering the Valley are placed under control, it is believed that the heavier silt following along the bottom of these streams may be segregated and conveyed in small flumes to reclaim much waste land which is now useless, such as the beds of old sloughs, low places that have become alkaline, and small areas of swamps which cannot easily be drained. In Italy and the South of France, large areas have been reclaimed by this method, or colmatage, as it is called, and much may be learned from their experience; but, in segregating this heavier silt, something may be learned from the intolerable placer miner, who, when he puts in his "undercurrents," as he calls them (for an entirely different purpose), points the way whereby the lower strata or heaviest silt, rolling along the bottom of a channel, may be diverted and carried away. The sketch, Fig. 4, shows the process better than it can be described. The secret of it is that the slot or exit orifice extends entirely across the bottom of the channel, and the velocity of the water through this orifice is not retarded by the sidewise discharge channel underneath. By a suitable device to vary the width of the exit orifice, any desired quantity of the heavier silt may be removed. The only mishap it is subject to in working is that it may get clogged by water-logged sticks and leaves rolling along the bottom with the silt. Opening the orifice wide for an instant will generally clear it. It is thought that a trap of this kind may be located in the heavier silt-bearing streams (they might be built into the flat portion of a barrier) where sufficient grade can be found, and convey the semi-liquid silt in small flumes to localities where needed.

DIAGRAM SHOWING OPERATION OF SILT OR SAND GATE

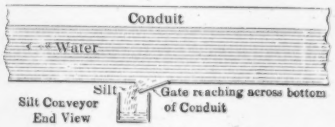


FIG. 4.

Finally, in working for the benefit of the tiller of the soil, the placer miner is reached. With the barriers in the streams to hold the

coarser part of the miners' débris, the finer silt goes to the soil, and the farmer gets more gold out of it, through his crop, than the miner in his riffles. It can be demonstrated in hundreds of instances among the foot-hills, that enormous crops are produced on soil made entirely from the "slickens" of the mines, and even the raw slimes from the stamp mills, collected in terraces in shallow gulches by little brush dams. It seems as if the raw rock, so finely ground, presented its potash and phosphorus to vegetation in a form that can be readily assimilated.

It is not probable that placer mining will ever become as large an industry as it was before it was stopped by law, but any placer mining will benefit the farmer, at the same time it will throw more or less coarse material into the stream beds, and this must be retained by raising the barriers oftener than would otherwise be required. It would be but reasonable, therefore, that the placer miner be taxed sufficiently to pay for the increased work required at the barriers, even if his silt does enrich the valley lands.

It is expected that the Sacramento River will rapidly deepen itself after the barriers are in place, yet there will be much dredging to be done, as shown in the Report of the Commission of Engineers; but, having the escape channels, as provided in this basin system, the river will never be required to carry much more than two-thirds of the greatest flood, and therefore it will only be necessary to enlarge the carrying capacity of the river to about two-thirds of that demanded by the Commission of Engineers, except below Cache Slough. From Cache Slough to the Bay, room must be provided for more than 300 000 cu. ft. per sec. for the Sacramento River, in addition to that required for the San Joaquin below the junction.

Assume the heaviest flood conditions ever known (or those intimated in the first part of this paper), as sure to come in the Sacramento Valley, and imagine the basin scheme complete. It is believed that the operation would be as follows: In the beginning of the flood, the regulating dams would be closed, the feeder gates open, and the basins filling. Ordinary judgment would imply the opening of the escape gates somewhat before the basins were full. As the floods rose, the regulating dams would open, to pass the excess down the rivers. The floods still rising, until all escape gates are wide open, all dams down, and the rivers full, there would still be room in the basins for

considerable excess, until their limit was reached. In this condition, the flood capacity of the river and escape channels would be somewhat more than 100 000 cu. ft. per sec., greater than was ever required in the Valley.

It is reasonable to assume that these conditions would not continue long, and would not occur later than March. Very soon the feeder gates would be closed and the rivers would be carrying the falling flood. The escape channels would drain the basins rapidly. Sowing could begin in the higher basins by the middle of April, and in all by the first of May. This may be thought to be too late for a good wheat crop, but it must be remembered that a heavy flood in the Valley means a heavy snowfall in the mountains and a long snow flood, so that a late sowing of the crops is compensated by a late irrigation, that is, that the feeder gates will be opened again near the end of June, and the crop will then get a heavy irrigation at the time of the hot winds of late June and early July.

From the foregoing it may be seen that a flood 25% greater than that of 1862 could pass through the Valley, not only without injury, but with vast benefit to the crop which would surely follow.

Assume the conditions in an extraordinarily dry season: Unfortunately, knowledge in reference to the weather is not yet sufficient to predict, at the beginning of the winter, what the rainfall is to be, but, in any event, it will always be well to begin turning the muddy water into the higher basins as soon as it comes; and, if there are no rains of consequence following, by closing the regulating dams, all the water there is can be utilized and spread as equably as possible over the basins and give them at least a short soaking. Early sowing will be made, and, as there is always some snow flood, it may be all used for the summer irrigation early in June by closing the dams again. In the extra dry year, therefore, the basin system will utilize all the water there is, and make something of a crop, though the land will not get much silt, and must draw on its previous enrichment to carry its crop.

It is conceded that this scheme, as presented, will cost many millions. On the other hand, it should be conceded that where the cost is millions the return is hundreds of millions.

A reasonable estimate for the average increase in the value of the land benefited is $100 per acre, and there are approximately 3 000 000

acres to come under the system. The increased value of the crop produced annually would be $20 per acre, an annual value of $60 000 000.

The value of the navigable waterways produced by the building of the system is very intimately connected with the value of the land, and may be included in that estimate.

The value of the flood protection cannot well be estimated in dollars, as it includes human life and happiness; but the increased value of real estate in Sacramento and Stockton will be of importance when it is once realized that there is no danger from floods, and that these cities are surrounded by millions of acres of ever rich wheat lands.

The renewal of placer mining, with its demand for supplies, will also contribute to the prosperity of the Valley, and the bitter war between the valley and the hills will end in both being benefited by the "slickens."

It will take a number of years to complete the scheme, and may require $75 000 000, which can be borrowed by the State as needed, and returned to the State by the lands benefited, in installments, after the benefit has been received, in a manner similar to that followed by the U. S. Reclamation Service. The benefits would cover, not only the lands in the basins, but also the cities and towns, and the mines, or all property in the Valley. The assessment might be arranged as a general tax on all property, sufficient to pay the interest; and, as basin lands would receive the most benefit (after they began to produce), a repayment tax of $2.50 per acre for ten years might be imposed, in order to pay back the principal. Afterward, a maintenance tax, sufficient to maintain the system, should be placed on all property.

DISCUSSION.

Mr. Harts.

WILLIAM W. HARTS,* M. AM. SOC. C. E. (by letter).—The problem of determining what treatment is best for the future prosperity of the great Sacramento and San Joaquin Valleys in California will require much patient study before anything entirely satisfactory can be evolved. The main future development of these valleys will undoubtedly be agricultural for many years, and whatever ministers most to this development as an ultimate benefit will unquestionably be the correct solution. The interests are so diverse and the difficulties of harmonizing them so great as to make the task one of enormous proportions. If the problem to be solved were the project for flood protection or for drainage of the valley alone, or if only the interests of navigation were to be considered, it seems safe to say that some definite plan could be readily devised which would satisfy the conditions fairly well. But the problem of harmonizing these important interests, difficult as it might be found, is further complicated by the necessity of providing for irrigation at certain seasons, by the improbability of securing perfect agreement among the many landholders, in order that the enormous expense may be distributed properly, and, further, by difficulties in the way of legislation, to the end that State and Federal interests may not be neglected, and, at the same time, that the charge for the necessary work be placed where it should properly belong.

Most of the lowland of this valley was given to the State of California by the General Government in 1850, under what is known as the "Swamp and Overflowed-land Act," with the distinct understanding that the State should take such steps as would reclaim these areas and render them fit for agricultural purposes. If this land were still "public land" of the United States, the solution of the difficulties would be far simpler. Practically all this land was sold later by the State to private parties at merely nominal figures, still coupled, however, with the obligation that it be reclaimed. In most cases this obligation has never been met, but decisions of the courts hold that the land is still subject to the call of the State for such assessments as may be found necessary for the proper solution of this great problem.

The entire central part of California consists of an elliptically-shaped valley of about 57 000 sq. miles, of which about 18 000 sq. miles are lowland, all having but one outlet, namely, through San Francisco Bay and the Golden Gate. The low part of this entire valley, it is believed, has been largely built up by the washing down of material from the surrounding mountain sides, until Suisun Bay, at its western side, is now all that remains of an enormous inland sea. It seems undeniable that the rivers flowing into this basin never have been, and probably never will be, able, in their natural condition, to carry off

* Major, Corps of Engineers, U. S. A.

within their banks the floods arising from the rains and snows falling within their water-sheds; and it also seems undeniable that the sediment eroded from the high mountain sides has been carried down probably for ages and is still moving seaward, encroaching yearly on Suisun Bay. Since the discovery of gold this erosion has been aided enormously by the process of hydraulic mining, millions of cubic yards of solid material having been dislodged from the gold-bearing areas only to be transported down the stream beds into the river systems below. Thus a further problem has been added to the natural difficulties, that of mining-débris control, the solution of which is now being attacked. Mr. Harts.

It may be true, as has been stated by some, that it is an "engineering absurdity" to attempt to control the maximum flow of the Sacramento River by levees, but, notwithstanding such hard words, it is the writer's belief that in no other way can the ordinary floods of this valley ever be satisfactorily controlled, and in no other way can the delicate problems of navigation, from which much good may be expected during the future development of this valley, be solved than by some system by which practically all the river flow shall be carried in the river bed. The width apart of the levees and their height determine the quantity of water which can be controlled. The slope is practically fixed. By deepening the natural bed, navigation and drainage channels can be provided; but the proper placing of the levees must be relied on to control the high-water flow of the river. Such extreme floods as it may not be economical to provide against must occasionally find an outlet by side channels, but the main river flow must be kept within the natural or artificial banks. Accessory works for drainage and basin irrigation may be easily coupled with this project, so that all that may be necessary in the way of flooding the side basins could be attained by the use of gates. A modified dike system, such as is proposed by the author, might be easily applied as an accessory improvement, but the main principles as given above will inevitably be found necessary in the end.

The requirements of navigation demand that deposit shall not be made in the channels of the stream. The plan proposed by the author would certainly produce such deposits. Even relief weirs in the levees cause a shoaling in the channel below, owing to the diminution in the currents in the main stream. It is readily seen that withdrawing a portion of the flow will cause a slackening of the currents below the point of the withdrawal, and deposit will surely result, to the injury of the channels. Side drainage channels will likewise be filled up by deposits therein, owing to the diminishing velocities. It is well known that the escape of floods over the banks causes the river beds to fill and the banks to build up in the case of all sediment-bearing streams.

Experience in many rivers and in many countries shows that, in

Mr. Harts.

order to secure the scour, to maintain not only navigable depths but adequate drainage, practically the entire flow of the river must be controlled between either natural or artificial banks. The improvement of the Po in Italy demonstrated this nearly two hundred years ago. The mouth of the Mississippi and the mouth of the Danube illustrate this point still further. It seems probable, therefore, that no project which does not include the deepening of the present bed of the river and the control of practically all of its high-water flow by levees can ever succeed. The author provides nothing for flood control. He accepts the present flood conditions as necessary and inevitable, and endeavors only to make the best of them. To anyone familiar with the high-water conditions and the slight fall of the main rivers of the valley, and the natural delay in passing floods, the problem will seem to be far from similar to that of the Nile, from which he draws his examples, and far from solution when he proposes to hold back by dikes, even longer than is done at present, the escape of the surplus water.

As to the details suggested by the author, some criticism should also be made. His suggestion of passing drainage channels under the Sacramento River by means of siphons seems particularly vulnerable. This river, flowing as it does through sedimentary deposit, would require work of tremendous expense to conduct side channels under its bed safely, and the probability is not remote that such siphons could be kept open only with the greatest difficulty. The available head must necessarily be slight. This is obvious when the great difficulty of drainage rests primarily on the lack of fall between the floor of the valley and mean tide level. Furthermore, the water comes charged with sediment which would unquestionably be deposited in these siphons unless expensive apparatus were devised to maintain the flow.

Mr. Foote suggests that fifty years ago the conditions of depth for navigation and drainage were far better than now, and were "of immense value to the community as commerce bearers." Records show beyond argument that fifty years ago, and even before that, the depths were no better than they are to-day. The bed has been raised slightly by sediment, but levees were built for the reclamation of the land, which have corrected this, and the navigable conditions are now no worse than they were then. Charts of 1841 show ruling depths of 7 ft. at low water between Sacramento and Suisun Bay. This depth is still available. At that time a 4-ft. depth was found between Sacramento and Colusa, and is still found. In the face of many difficulties, the former navigable conditions have been maintained to the present time. If the rivers are not as important now as they were then for carrying commerce, the answer is to be found in the development of the railways. That river commerce was greater then than now is often asserted, but never verified, and is open to challenge.

Attention is invited to an oversight in the assignment of the credit

Mr. Harts.

for the design of the movable dam mentioned by the author as a desirable type for construction on these streams. The dam, referred to as having been designed by Mr. E. R. Cooley and used by Isham Randolph, M. Am. Soc. C. E., at the locks and power plant of the Chicago Drainage Canal, was originally invented by H. M. Chittenden, M. Am. Soc. C. E., Col., Corps of Engineers, and used by him in the improvement of the Osage River.* It is a modification of the Desfontaines drum wicket.†

Another detail requiring comment is the author's method of consolidating gravel and sand by pumping this aggregate full of cement grout. While engaged on the restraining works on the Yuba River, the experiment of pumping Portland cement grout into the bed of the river was tried with a view to consolidating it. Many experiments were tried, but all proved unsatisfactory. The first and main objection was that one had no definite knowledge of the results of his work, and could not learn without more expense than other methods would cost; the second objection was that the water encountered affected the setting qualities of the grout, through dilution or otherwise; and third, it was found impossible to force the grout to any great distance from the pipes. The experiments on the Yuba River, where conditions were peculiarly favorable, showed conclusively that no reliance could be placed on this method. The writer does not mean to say that there may not be occasions where it might prove advantageous, as a matter of fact, it has occasionally been attempted successfully; but the conditions in the Sacramento Valley are somewhat similar to those on the Yuba River, although not so favorable for this class of work, and he believes that the author is over sanguine as to the results achievable in this way.

The holding back of the débris by a succession of overfall barriers, in the plan suggested by the author, has been abandoned by the California Débris Commission after thorough trial, and is open to serious objection. This method was applied years ago to the Yuba River, and several failures have shown that overfall dams in the beds of rivers composed of mining débris cannot be maintained without undue expense. The first attempt of this kind was made by the State of California in the early Eighties. A barrier of brush, extending entirely across the Yuba River, was built at a cost of about $250 000. This work was designed by the State Engineer, with the approval of a commission of consulting engineers, of which the late James B. Eads, M. Am. Soc. C. E., whose reputation for river work was widely known on account of his connection with the improvement of the mouth of the Mississippi, was a member. This barrier did not resist the first freshet. The next attempt was made some years later by the California Débris Commission farther up stream, and was also a brush and rock

* *Transactions*, Am. Soc. C. E., Vol. XXXIX, p. 533.

† *Journal of the Association of Engineering Societies*, June, 1896, p. 249.

Mr. Harts. dike combined with sheet-piling on the down-stream side. This dike was likewise destroyed, and later a modified design was again attempted, using brush fascines built in "cob-house" construction and filled with rock. This plan also failed. It was evident that, even with low lifts, overfall dams on beds of gravel could not be attempted in this river, and they were then abandoned by the Commission. Accordingly, a later type of barrier was designed, to be built of concrete in successive steps held in place by piles, the down-stream side in the river bed to be composed of a broad apron of heavy rip-rap, the main flow of the river to be carried through a spillway cut through solid rock at one end, so that at all stages, except at extreme high water, all floods would pass through a rock cut, leaving the dam and the bed of the stream untouched. While under construction, and before this barrier was completed, it was partly destroyed by the extreme flood of 1907, by wearing away the concrete cover, and has not yet been rebuilt. This same general method is now being followed at Daguerre Point, lower down on the Yuba, where a dam, too high ever to be overtopped, and a side spillway, will impound more than 50 000 000 cu. yd. of mining débris. The plan suggested by the author of placing various overfall barriers across this and other similar rivers is open to this objection. No overfall dam can ever be maintained in these streams except at undue expense.

Finally, the enormous cost of any such work as proposed by the author, even if it should be assumed to be effective, would probably forbid its application. In 1905 a comprehensive plan for the drainage of the Sacramento River was prepared by a board of experienced engineers skilled in river work. The cost of this project was about $24 000 000. The question at once arose as to how this money could be secured. The Federal Government was appealed to in the hope that it would pay one-third of the amount. The State, it was hoped, would pay one-third, leaving the remainder to be paid by the landowners interested. Inasmuch as many of these landowners had purchased their lands for $1.25 per acre, and some had had even this amount refunded to them by the State, it would appear that their hope of having two-thirds of the expense of reclamation borne by others was, to say the least, not an unselfish one. It is needless to say that thus far neither the United States nor the State of California has accepted any such disadvantageous proposition. It seems clear that the first difficulties to be surmounted in any adequate plan of treatment would be a proper distribution of the expense under such legislation as would compel all the improved property to pay its proportionate share toward the improvement. That the greater burden of this work must fall on the land in question there seems no doubt, but no project can be undertaken until some just method of collection of funds for its completion has been provided for by law.

H. H. WADSWORTH, M. AM. SOC. C. E. (by letter).—If the plan proposed by the author could be carried to completion, it might transform some of the serious evils resulting from frequent floods to substantial benefits, but its principal merit, if it possesses any, lies in its irrigating possibilities. Mr. Wadsworth.

Before discussing briefly some features of this plan, the writer feels that several of the author's statements should not be allowed to go unchallenged.

The statement that "Fifty years of mishandling natural riches and spurning natural laws have so far injured it that now it may be said, in an economical or engineering sense, the Great Valley is lost to the world" cannot be taken seriously when the population of the valley and the already great value of its products are increasing. No doubt the raising of any one crop for many years consecutively is a great drain on the fertility of the soil, but if "the historic wheat fields have deteriorated until they are no longer of appreciable value to the State" as wheat fields, it does not follow that they are not of much greater value for the production of crops which give much greater returns on the capital and labor invested. What the limits of the Davis farm were, the writer does not know, but, to him, the country lying about Davisville and northward to Woodland, and beyond to the land in Colusa Basin, from which last winter's flood did not recede in time to permit of a crop being raised, presents an appearance very different from that which the author's statements would lead one to expect. Even if such yields of wheat as the virgin soil produced could still be raised, the soft variety of wheat grown in such climates as that of California, on lands worth from $100 to $300 per acre, could not compete with the No. 1 hard wheat of the Dakotas and the Canadian Northwest, grown on lands worth from $5 to $50 per acre.

The author's statement as to the present condition of commerce on the Sacramento and San Joaquin Rivers is also very misleading, as these rivers are still of great value to the State as commerce bearers. The commerce on the Sacramento and San Joaquin, between San Francisco and Sacramento, and San Francisco and Stockton, respectively, has increased in recent years, though naturally not in the same ratio as the population, as much of the commerce can be handled much more satisfactorily by railroads. A navigable depth of 6 ft. is maintained to Stockton and to Sacramento, and 4 ft. to Colusa. Even above Colusa, where, during the low-water season, the author's statement that "a few flat steamboats and barges struggle from one sandbar to another" might be applied (though not to the lower river as he states), conditions are better than they were twenty years ago. From tales of steamboat men, with whom the writer has talked, and who have sailed the river for fifty years, conditions on the upper river, as long

Mr. Wadsworth.

ago as that, were not very different from those now existing. Much is heard of ocean vessels having reached Sacramento in the early days, but it should be borne in mind that ocean vessels of that time were of very much lighter draft than those of to-day; also, that in June, 1908, a fleet of several ocean-going torpedo boats of the United States Navy, made the trip to Sacramento without difficulty. In 1908, 400 000 tons of freight and 150 000 passengers were transported on the Sacramento River. This amount of freight is about equal to that carried, during the same period, on the Mississippi between St. Louis and Cairo, and is about one-half as much as that carried on the Mississippi between Cairo and Memphis, exclusive of the coal shipments, which alone amounted to more than 1 000 000 tons. These shipments, however, are in a class by themselves, being made in barges which are floated down from Ohio River points when a favorable high-water stage occurs.

The "fifty years of mishandling natural riches, and spurning natural laws," however, has resulted in abolishing navigation almost entirely on some of the Sacramento's tributaries and greatly injuring it on the main stream; also in greatly restricting its flood-carrying capacity.

This latter effect is the result of two actions: First, the deposit of vast quantities of mining tailings in the tributary streams; and, second, the irrational procedure in the construction of levees for land reclamation. Since the cessation of all extensive hydraulic mining operations, the evil resulting from the first of these causes is correcting itself, aided to some extent by restrictive works. The low-water surface of the Sacramento River is now lower than it has been for many preceding years, and still the available navigable depths are greater. The lack of system by which land owners or reclamation districts have located levees to suit themselves, and by which those having the greatest resources have been able to maintain their levees above flood height by the simple, but costly, process of making them enough stronger than those of their neighbors so that the latter would give way first, has resulted in contracting the natural waterways so that they are incapable of carrying, in some instances, more than 20% of the flow naturally tributary to them, even with an hydraulic grade line up to the top of the highest of the existing levees, thus forcing the flood to seek the weakest point and spread over the adjoining low lands. As long as the Cities of Sacramento and Marysville are able to maintain levees stronger than, and well above, those on the opposite side of their respective rivers, their "luck" in escaping destruction will be likely to continue. This same lack of system, which has resulted in leaving so little space between the levees as to reduce greatly the flood-carrying capacity, has, in general, resulted in an improvement of conditions as far as navigation alone is concerned, and, for this reason, makes it

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desirable to obtain some solution of the flood problem other than the widening of the space between the levees to the extent of taking nearly the whole strip of already reclaimed land. This would be the case in some places if the plan, recommended by the State Commission of 1904, to enlarge the strip to provide for floods 100% greater than the maximum assumed by it, were followed. The floods of March, 1907, and of January, 1909, have shown such an increase to be necessary.

The author's plan would still enable the great flood basins to perform their functions as storage reservoirs, which they do very effectually now, but only at the expense of great wash-outs in levees and the flooding of valuable rim lands over which the water must pass in order to reach the basins. This plan of holding the flood waters in elongated, stepped, settling pools, although very expensive, would, like the plan for storage reservoirs in the mountains, relieve the flood situation to the extent of the storage capacity afforded. It would also have the advantage over that plan, as far as flood protection goes, that such reservoirs could be kept empty in anticipation of a flood without subjecting the lands dependent on stored water for summer irrigation to the hazard of a water famine, if the last flood of the season should be allowed to escape in anticipation of another.

As to the capacity of this scheme for holding back flood waters, some figures made by the writer, based on rather uncertain data, show that if the entire area of the lands now subject to overflow, together with sufficient additional border land to compensate for decreased depths in the troughs of the basins, were available for the purpose, they would hold the mean flow of the Sacramento and its tributaries, when equal to that of the March, 1907, flood, for about 3 days, while the mean discharge mentioned continued for 4 days and was much in excess of the channel capacity for several days longer. It may be thought at first glance that the area covered by the irrigation basins, as shown on Plate XXII, is so much larger than those which have been overflowed in the recent great floods that they would more than compensate for the decreased depths which the stepped basins would give; but the relief from flood conditions is dependent, not only on the total cubic contents of the storage basin, but also on the rate at which the water can reach it. The high-level basins most remote from the river would fill so slowly that they would afford very little relief in floods so flashy in character as those of the Sacramento. It is evident, therefore, that with this system in operation, a flood channel, of great capacity, to the sea would still be necessary. The carrying out of the basin irrigation plan involves the co-operation of the owners of very much more property than does that of a straight flood channel, and the cost would evidently be so enormous that it would seem to be the part of wisdom to construct first, works to relieve the flood situation

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as simply and economically as possible, though at best these will be complex and expensive enough.

The basin irrigation project could then be developed along lines similar to those of ordinary irrigation projects, and step by step, as fast as circumstances warranted. When the system is complete, portions of the land previously used as flood channels can be diverted to agricultural purposes, if found practicable.

The discussion of the relative merits of basin and perennial irrigation, the writer will leave to others. A recent article on irrigation in Egypt indicates that basin irrigation is being discarded for the other kind. As to the feasibility of carrying the silt-laden waters to the extreme points of the higher basins and of depositing silt in them for the purpose of fertilizing the soil, that is, of course, dependent on the velocity of the water and the time which can be given to the finest of the silt to deposit. In view of the fact that, along both the Sacramento and Feather Rivers, the immediately adjoining ridges and slopes, on which most of the material carried in suspension is deposited, are the choicest of agricultural lands, while the troughs of the basins (particularly the upper parts of Colusa and Butte Basins) are comparatively sterile, alkaline lands, it would seem open to question as to whether much fertilizing material is deposited on the latter, although they are under water for considerable periods each flood year.

It is a growing conviction of the writer, after two years of observation, that the agricultural conditions of that part of the Sacramento Valley above the mouth of the Feather would have been much better than at present had no levees been built above that point.

This would have allowed the rising river to spill uniformly over its banks in a thin stream instead of being held by levees to a great height finally to break through and wash out a great hole in the natural ridge on which the levee is built, well down toward the bed of the river, through which break enormous quantities of sand, as well as the more desirable silt, are carried to the adjoining land.

Well-directed efforts at drainage might then have unwatered large portions of the basins in time to grow quickly-maturing crops, such as are now grown on many acres of land lying between the levees and the river.

The many structures required by the proposed plan will, as the author points out, involve a high order of engineering skill, and some of them, such as the siphons to carry one river across another and those required in the treatment of other rivers as miners' sluices with "undercurrents," will certainly be monumental.

The use of cement grout injected into natural sand and gravel deposits for forming foundations is doubtless worthy of great extension, but should be subject to the greatest care. During the fall of 1905 the

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writer, with F. E. Frey, Assoc. M. Am. Soc. C. E., made some experiments along this line on the Yuba River, immediately below the toe of the débris-restraining barrier. The record of these tests was destroyed in the San Francisco fire of the following April. As now remembered, six 2-in. pipes, 14 ft. long, and spaced 3 ft. apart in a straight line, were driven by a water jet supplied by a powerful steam pump. Owing to the character of the formation, the material ranging from fine sand to cobbles and boulders, it was very difficult to secure the desired penetration, although it was finally obtained to within a foot or two in each case. The same difficulty was met in driving piles with a jet in this vicinity.

Cement grout was then forced through the pipes by a hand-force pump, the pipe being raised slowly as the grouting continued. About one bag of cement was used for each foot that the pipe was raised, this being the limit of quantity that could be forced through the pipe until after it had been withdrawn about one-half its length.

As the lower end of the pipe neared the surface, the grout generally boiled up through the gravel in a concentrated stream which followed the line of least resistance to the surface. In one instance direct communication was established between the bottoms of adjacent holes, the grout being forced down one pipe appearing at the surface through the hole from which the preceding pipe had been drawn. If several pipes could have been put under pressure at once, no doubt a more uniform distribution of cement would have been secured.

During the winter following the grouting experiment a flood washed away the gravel to the depth of several feet, and broke off this artificial cement dike. Some large fragments of the concrete, measuring from 2 to 4 cu. ft., were found some 40 ft. below the point where they were made. These were of good sound concrete, but showed, what had been felt to be the case when the grouting was in progress, that the penetration had been very irregular, that in order to secure good massive concrete, pipes could not be placed farther apart, in any direction, than in this experiment, and that several pipes should be worked at the same time in order to secure a fairly good distribution of the cement.

With gravel and sand deposits of a more homogeneous character, and with a well-appointed plant, very satisfactory results can doubtless be obtained, but the construction, by this method, of a suitable foundation for an important structure in a river bed consisting of sand and gravel to the depth of 60 or 80 ft. would, it seems to the writer, be somewhat uncertain.

The one thing most needed for the improvement of conditions in the Great Valley of California, before the various interests involved can be induced to work together, seems to be a campaign of education. Within the past few days an attorney in a suit on trial in a County

Mr. Wadsworth. Court in the Sacramento Valley maintained that "the common law rule gives the land owner the right * * * to protect his land against flood waters without regard to the damage his methods may inflict on others." This may be good law, but the continued observance of this principle will prevent the execution of any comprehensive plan for the reclamation of the lower lands of the flood basins and for the relief of the higher lands from the periodically flooded condition.

Mr. Veuve. ERLE L. VEUVE, M. AM. SOC. C. E. (by letter).—The remarkable plan proposed by Mr. Foote for the redemption of the Great Valley of California, as he terms the basins of the Sacramento and San Joaquin Rivers, is certainly bold in its inception and, owing to the importance to Central California of reclaiming the swamp and overflow lands along these rivers and of preventing disastrous floods in the future, his paper should arouse considerable interest among the California members of the Profession. The plan proposed is of great magnitude, and will cost an enormous sum of money, but, when one considers the conditions along these rivers at times of destructive floods, such as experienced in 1907, and the enormous losses sustained, it would seem that the people of the Valley, backed by the State, could readily undertake such a task provided it could be shown with reasonable certainty that it would bring about the desired results and also the financial gains claimed by the author.

Mr. Foote states that any plan proposed to relieve the situation must take care of the various interests—navigation, flood protection, drainage, irrigation, mining débris—or else arouse a great deal of antagonism from such interests as may be slighted. He states that:

"It is believed that 'the solution of the problem for the tiller of the soil' is in harmony with and solves all other problems in the Great Valley."

But, is the problem as simple as this? Assuming that by solving the problem for "the tillers of the soil," all the interests specified are cared for, are not the interests of the farmers themselves antagonistic? Are the interests of the alfalfa and fruit grower identical with those of the man who raises wheat and grain? Would the orchardist take kindly to the basin system of irrigation with its resultant depth of from 2 to 9 ft. of water standing in his orchard?

The Miller and Lux Corporation owns many thousands of acres of land, extending for eighty miles along the San Joaquin River. It has valuable water rights and many miles of canals. Many acres are devoted to raising alfalfa, and large areas are fenced off for cattle ranges. Cattle are shipped from outside the State to run these ranges during the winter months. Alfalfa is used to supplement the range feed and to fatten the cattle for market. Would it be to its interests to have the cattle ranges flooded at a time when they are of the greatest value?

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Likewise, considerable areas in the San Joaquin Valley are proven oil-bearing lands, and new fields are rapidly being discovered and developed. Land in these fields is valued anywhere from $500 to $5 000 per acre. Would any scheme of basin irrigation harmonize with these interests?

The question which naturally presents itself is whether or not the entire valley should be turned over to the raising of grains and such crops as would adapt themselves to basin irrigation, or whether, in certain localities, it would be practicable to segregate the land which would be more valuable for other crops. And, finally, is not the area which is useful for the more valuable crops too great and too valuable ever to induce the owners to consent to the plan suggested by Mr. Foote? These few points have occurred to the writer, but no doubt the author has thought of them and has in mind a solution.

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FREDERICK HALL FOWLER, JUN. AM. SOC. C. E. (by letter).—The writer having recently been in Egypt, studying and observing the irrigation system, wishes to point out some radical differences between the operation of the "basin system" in that country and in the Great Valley.

The greatest difference is in the origin of the floods in Egypt and California, with regard to their respective "basin systems." In Egypt there are two floods, which follow each other closely. The first, and greater, originates on the Abyssinian plateau, and reaches Egypt from the Blue Nile and River Atbara branches. The second, and lesser, originates on the Lake water-shed of Central Africa, and along the Sobat River, and reaches Egypt from the White Nile branch of the main river. The important point to note is that the "basin systems" of Upper and Middle Egypt receive their flood waters from floods that come a very great distance, the country itself being almost rainless.

Since the capture of Khartoum from the forces of the Mahdi, there has been telegraphic communication between that point and Egypt. This has given the greatest advantage in the working of the basins, for it takes a flood 11 days to travel from Khartoum to Assouan; and, from Assouan to Cairo an additional 6 days are consumed. The operators of the basins now have, therefore, a minimum of about 12 days' notice of the flood conditions, and can govern themselves accordingly. To be sure, this is a very recent advantage, but it is a very great one.

In the Great Valley of California, the conditions are far different. The flood originates on the immediate borders of the water-shed of the basins, and is most likely to come to a peak at the very same time that the heaviest rains are falling in the basins. Moreover, above the basin system the grades in the flood plane increase rapidly, giving the flood a character which is almost torrential. In addition to this, the

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very highest floods often occur during the last storm of the rainy season and after a prolonged season of heavy rains. The weather conditions directly over the area of the basins are at their very worst.

If the basins are to be used for the irrigation of the Great Valley, the scheme of using them as flood moderators is at once defeated, for it would be very unwise to leave the basins empty until after March 15th, in anticipation of using them as escapes for a flood which might not arrive until the next rainy season. In short, the combined use of "basin irrigation and basin flood storage" cannot well be applied under the conditions to be found on the water-shed.

If, in order to secure the best results from the basin lands of Upper Egypt, the basins have to be discharged on top of a flood that has lasted longer than usual, the result is often disastrous to the country from Cairo to the sea. Several times severe destruction of property has been avoided solely by a kind of automatic release, the river bursting its banks, flowing into one of the lower basin systems, rupturing the transverse dikes, and using the basins of the system as a by-pass to a lower point where the flood congestion is not as great.

Judging from the experience in Egypt, it would seem to the writer that the best result in flood protection for the Great Valley would be obtained by adopting, as far as possible, the following programme:

1.—Straighten and improve the present channels, so as to bring them to their greatest carrying capacity;
2.—Design the levee system on both banks to carry as large a flood as possible "within banks";
3.—Then use the basin system as a by-pass, to handle unusual floods.

In a basin system used for this purpose, the inlets and the escapes would have to be proportioned very generously, and the drainage of the basin, and its carrying capacity, improved to the uttermost. During the highest floods, the rivers of the Great Valley are now taking by force what they cannot get by engineering help, and, at the peak, large crevasses are formed, which provide escapes for the congested channels. It would seem better to build a system of escapes so that this discharge could be handled with some degree of certainty, without waiting for the rivers to seek their own courses through the lines of least resistance.

One of the greatest weaknesses of the "basin system" is that it is wholly dependent on the profile of the flood. The working of the system will approach its greatest success only when the flood is of the proper height and the proper duration. The flood must rise slowly enough and remain above a given height long enough so that the basins may be filled properly. It must drop fast enough, and at the right time, in order to allow them to be emptied and drained properly. Any varia-

tion from these conditions will produce a proportional reduction in the efficiency of the system. It is doubtful whether the flood wave of the California rivers is of the proper duration. Mr. Fowler.

The whole tendency of Egyptian irrigation at present is to change, as far as possible, from "basin" to perennial irrigation. Even Willcocks, whose praises of the "basin system" are quoted by the author, has changed his views on the subject, and says:*

"That basin irrigation, which has been typical of the country for 7 000 years, is giving place everywhere to perennial irrigation, and the current is so strong that individual views are being abandoned, and from one end of the country to the other there is an eager demand that double crops per annum shall replace the ancient single-crop system. The science of manuring and rotating crops on one hand and the practice of draining and irrigating by rotation on the other, have made such rapid and simultaneous strides, that lands can now be made to produce their two and even three crops every year and still retain their full vigour."

The writer has not made a very thorough technical study of the sediment deposited on the flooded areas by the rivers of the Great Valley, but it seems to him that, in fertilizing qualities, it is far inferior to that brought down by the Nile. The red water of the Egyptian basins is the wash from the lava of the Abyssinian plateau. From the nature of the water-shed, the distance which the floods travel, and from their rather moderate velocities, it is reasonable to suppose that only the finer particles of their contents remain in suspension. In the California rivers the case is different. The floods are more torrential in character, traveling relatively short distances at higher velocities, and coming from a water-shed which, in addition to being more granitic and coarser in formation, has recently been denuded of a large part of its forests, and badly cut up by placer mining operations. The silt deposited by this flood must necessarily be less rich, and more in the nature of débris. In discussing this matter with those who have been engaged in the reclamation of tracts in the Valley, the writer has been told that this is the case in a number of instances. The deposit is of a poorer grade than had been anticipated.

It seems to the writer that the soil exhaustion of the Great Valley can best be met by the proper rotation of crops, by fertilization, and drainage, under a system of perennial irrigation.

GEORGE L. DILLMAN, M. AM. SOC. C. E. (by letter).—Mr. Foote has taken for his subject the greatest engineering problem of the Pacific Coast, those connected with the transcontinental railroads and Sierra Nevada water supplies for the Coast cities being small in comparison. The solution of this problem affects the State of California most vitally. These floods inundate the railroads, submerge and destroy Mr. Dillman.

* "The Nile Reservoir Dam at Assuan."

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thousands of acres of annual crops, affect the fruit and cattle industries, and threaten and sometimes flood the towns. The knowledge that they may come prevents the cropping of thousands of acres in the "basins." Butte, Colusa, Sutter, Yolo, and American Basins contain more than 300 000 acres of land which, at present, is almost useless. If reclamation were complete and these lands were available for crops, they would be worth, at a minimum, $100 per acre, or would add to the wealth of the State, say, $50 000 000.

Mr. Foote's theory is very beautiful. To capture the flood waters, hold them on the lands of the valley until they deposit their wealth of sediment, and then pass the decanted water on toward the ocean, is certainly ideal; but, what about its feasibility? The silt-carrying capacity of a stream depends on its velocity; and, with a fixed slope, the velocity depends on the volume. Decreasing the volume by side diversion, as proposed, would result in a deposit of silt in the main channels, which would raise the water plane of the floods. Therefore, each succeeding flood of the same volume would reach a higher mark, so that the areas submerged by the flood, instead of being the lower portions of the valley, would extend to higher elevations, and ultimately to the whole valley. The present tendency is in that direction, and the proposed construction would hasten the final result—the overflow of the whole valley.

Levee construction can be attacked on various grounds, but it certainly tends to keep channels open by decreasing the deposit of silt. Every big break in the levees has resulted in a material deposit of silt just below the break. The artificial diversion proposed would have the same effect.

Some years ago the State Board of Works proposed an elaborate system of by-passes for the regulation of the Sacramento, of which it was said, "No more comprehensive plan could be adopted for the ruin of the agriculture of the land and the navigation of the streams." The proposed plan seems to contradict this. The ruination would cost more and come sooner.

The escape channels are certainly "of first importance," as stated. Take the Yolo Basin: As the whole volume of the Sacramento and Feather Rivers at flood now goes that way, the cost of this channel would be considerable, and it would have to be re-excavated after every flood. The question of a drain channel to take only flood and seepage waters has appalled everyone who has considered it seriously. The floods could be passed between levees properly placed, but the basin system would demand an excavated channel, as the levees would submerge the large areas back of them.

At one time Tulare Lake undoubtedly drained into the San Joaquin River. Now it is a sink, formed by the deposition of silt from Kings River and Los Gatos Creek. The causes which formed it would fill

any artificial channel for draining the lake, or would entail enormous expense in its maintenance. Mr. Dillman.

As to cost: The one thing definite is the size of the levees proposed. These are 9 ft. high, 8 ft. wide on top, with slopes of 2 to 1, and they are to be placed 5 ft. apart in elevation. Taking an elevation of 200 at each end of the valley, their lengths would average 300 miles. This makes 12 000 miles of levees, containing roughly 550 000 000 cu. yd. The preparation of the foundations, as proposed—puddling the cores as the levees were built—would make the cost certainly not less than 10 cents per cubic yard, or $55 000 000 for the main levees. Allowing $5 000 000 for cross-levees, the cost would be $60 000 000 for "checking up the land" alone. In nearly every case the diversion dams would have to be built on recent alluvium. Squirting cement grout may or may not create a foundation; in the writer's experience, such attempts have been failures more often than successes. Taking it for granted, however, that the dams could be constructed cheaply, there would be several thousand of them. The number of diverting dams on the main river would be not less than 250, and a dam must also be built across every considerable drainage, at which points no doubt an escape or waste-gate would be demanded for drainage. The levees or checks would merge into dams at all such points. Although there are no data for estimating, these diverting dams, waste weirs, and dams across auxiliary drainages would cost, say, one-half as much as the levees, or $30 000 000. Then, the drainage system would cost certainly not less than the levees, or $60 000 000 more. Of course, these estimates are based on data insufficient for accuracy, but certainly not less than $150 000 000 would be spent on the proposed plans before their completion.

As to operation: This must be through human agency. The floods may come in January; they may come in March; they have been known to come later. These basins will be occupied by farmers, fruit raisers, stockmen, and others. Each man would prefer that the basin of his neighbor be filled rather than that he be discommoded by an artificial flood under the control of someone he can reach. There would be considerable friction, to say the least. Men will sometimes fight for sentiment, more will fight for property, but all will fight for their homes. Operation would be impossible, unless the land were under one control, which in itself is an impossibility.

As to maintenance: The annual cost would be a large percentage of the first cost. Breaking levees, silting drain channels, deterioration of wooden structures, the silting up of the land, requiring the checks to be raised, all argue a large maintenance. To these reasons—wrong hydraulic principles, prohibitive first cost, impracticable operation, and excessive cost of maintenance—the writer would add others, as grounds for differing from the author.

Only a part of the 3 000 000 acres is subject to overflow. The man

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with high land is not going to pay for the benefit of having his land flooded for the convenience of his neighbor on the lower land; he is going to be paid for it. To him it may or may not be a benefit; it is a matter of opinion until the fact is developed. The man on the lower land cannot pay for the reclamation except by mortgaging the increased product of the land resulting from the protection. No two men will agree on the method of procedure. The impracticability of organizing the project is evident; and, from what the writer has seen of the people of the valleys, he knows that the task of educating them up to a good project is well nigh impossible. To get them to agree on such a project as the one proposed is entirely out of the question.

The navigation interests, guarded so jealously by the United States Corps of Engineers, would be antagonized, and the result would be absolute obstruction there.

One point of difference between the floods of the Sacramento and those of the Nile is that the latter deposits its heavy burden largely above the agricultural lands. The silt brought down is finely divided, largely buoyant, and is carried in all parts of the depositing current in considerable uniformity. The silt of the Sacramento is heavy, is much more than mere silt, containing, at first, gravel, which is rolled along the bottom by the current, then sand, which is deposited *en route*, some of it being carried to Suisun Bay, or farther. The percentage of silt varies largely in different parts of the stream, increasing with the depth.

There is a plan of procedure which has appeared to the writer to be feasible. The waters of the Sacramento and Feather Rivers go down the Yolo Basin at floods. The Sacramento River at and below Sacramento has hardly sufficient capacity to take the American River alone. In substantiation of this, it may be stated that the floods of 1907 and 1909 each developed an up-stream current in the Sacramento River at Bryte's Bend.

The writer proposes a channel through the Yolo Basin to Gray's Bend, which will pass the flood quickly. This will lower the flood-water plane materially at Knight's Landing and in the Colusa and Sutter Basins, and will relieve Sacramento from apprehension for some years to come. It will cost much less than to get the necessary capacity in the present channel. It occupies land of small present value. All landed interests along the river would be benefited except at Sherman Island, and by strengthening its north levee, it would not be injured, and this should be part of the detailed plan. The benefit to Butte, Colusa, Sutter, and Yolo Basins would be direct and immediate.

The writer would propose to prohibit the construction of all other by-passes (only advising this on the grounds of expediency), and to vest authority in the hands of a competent board of engineers which would control the location of all levees and the manner of their con-

struction, and would see that all channels were controlled so that in the future they would have capacities sufficient for their flood volumes. This would follow the great American principle, the greatest good to the greatest number. Nothing would be adversely affected except the navigation interests. The railroads would have the floods about at the points where they are periodically washed away now, the farmers from Suisun Bay to Colusa and Marysville would be benefited, and Sacramento would not depend on chance for safety. Mr. Dillman.

C. E. GRUNSKY, M. AM. SOC. C. E. (by letter).—Having been called on several years ago by the Director of the United States Geological Survey to express views on the problems involved in the betterment of conditions in the Sacramento Valley, the writer reported in part as follows: Mr. Grunsky.

"The swamp and overflow lands of the State of California were ceded to that State by the United States in 1850 for the purpose of reclamation. These lands in California, as in other States, were disposed of to private parties under laws intended to encourage their drainage and reclamation, either by individual owners or by co-operation of owners under district organization when a number of tracts owned severally appeared susceptible of reclamation by one system of works.

"Under these laws, attempts to reclaim lands along the larger rivers have been mainly directed to the construction of embankments or levees, which serve as barriers to the spread of the water. Protection work of this character has generally been constructed at the sole expense of land owners. Some State aid has been extended, as in California, and in some cases, as along the Mississippi River, the work done by the United States has been a material feature of the land protection work. The measures thus far provided have led to only partial success. Proper attention has rarely been given to the drainage problems. The works for local benefit have gradually constricted the natural waterways and submersible areas, and this reduction of waterway, which has in some cases been aggravated by other causes, has forced the flood plane above its original height and has added to the difficulty and expense of further land protection. This is the situation in California at the present time. The natural main drainage lines, though reinforced by levees, are inadequate to carry the floods to the sea. Hundreds of thousands of acres are still submersible at ordinary flood stages, and other hundreds of thousands of acres, though in a measure protected, are subjected to dangers which make their cultivation possible only at a great risk. These lands are all privately owned. It is not a question of improving a part of the public domain.

"Harmonious action by the land owners has in the past been out of the question, mainly due to the vastness of the areas which should be included in single drainage projects. Those who believe themselves favorably located prefer a partial protection, according to their own ideas and at their own expense, to a participation in one comprehensive project. Disasters have been so frequent under this system that efforts

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are being constantly put forth for relief. One project after another has been suggested, but thus far without satisfactory result. There seems to be general agreement that the main difficulty lies in harmonizing conflicting ideas relating to the best course of action.

"It seems reasonable that rivers, such as the Sacramento River, which lie entirely within a single State, should be entitled to the same consideration by the Federal Government as the rivers, such as the Mississippi, which are inter-State streams, and it would be eminently appropriate and proper for Congress to adopt the same general plan of treatment for rivers of the former class, thereby aiding and facilitating the work of land reclamation.

"The impression seems to prevail among the interested parties in the Sacramento Valley that the United States may upon a further investigation undertake a plan of river rectification on a larger scale than contemplated in the past, which, while improving the navigability of the waterways in the Sacramento Valley, will at the same time fit in with a general drainage project and materially reduce the cost thereof.

"The view that further investigation, looking to a broader treatment of the question, is justified, led Congress to include in an Act approved March 3d, 1905, the following:

" 'The Secretary of War is hereby authorized and directed to appoint a board consisting of three engineers of the United States army (one of whom shall have had experience on the Sacramento River and two on the Mississippi River) for the purpose of making a general examination of the Sacramento, San Joaquin and Feather Rivers, California, and their tributaries, and of consulting with any engineers, commissioners, or officers who have been appointed by the State of California to determine the method of controlling the overflow of said rivers and their tributaries, with a view of considering what, if anything, the United States can or should do in conjunction with said State to improve the navigation of said rivers and their tributaries, and the probable cost to the United States of such improvement.'

"The Sacramento Drainage district has been formed by legislative enactment, aproved March 20th, 1905. The law creating this district, in making provision for the assessment of the property in the district to meet the cost of works, proportioned by benefits conferred, conditions this assessment upon a contribution by either the State of California or the United States, or by both. By suitable authority 'the proportion to be borne by said district and the State respectively of the cost of constructing and completing the work' is to be determined. No part of the district assessment 'shall be called or collected until the State of California and the Government of the United States, or one of them, shall have made an appropriation or other legal provision for the payment of the balance of the sum to be expended jointly with said district in performing the work according to the plans recommended by said report of said engineers, or such supplemental, amended or other plan as may be approved by the State Board of Examiners.' The report here referred to is the report of Engineers T. G. Dabney, H. B. Richardson, H. M. Chittenden, and M. A. Nurse, filed with the Commissioner of Public Works of California on December 15th, 1904.

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"Whatever is done by the United States or by the State for the amelioration of conditions as now prevail in the submersible areas adjacent to such rivers as the Sacramento River will result primarily in direct and material benefit to the owners of the land. It will result in indirect financial benefit to the State and to the United States, and in general benefit to the Nation, by adding to the resources of the country.

"The report of the Board of Engineers which has been authorized as above stated may lead to an appropriation by Congress, which can be used in the construction of works for the improvement of drainage and navigation in Sacramento Valley. If it does, and if by such means the drainage district is enabled to make progress along the lines marked out by the State law, there will be no call from Sacramento Valley for such assistance in the matter of drainage as it is believed could, with benefit to all concerned, be extended, if Congress should pass laws for reclamation by drainage similar in their scope to the Reclamation Act.

"But, in the other event, further aid from the United States will be sought—and the time will soon be at hand when a course of action in such cases must be definitely marked out.

"A wise, safe and sound policy to be adopted by the United States in cases where land is to be reclaimed by drainage would be to assist:

"First, by outlining drainage projects and supervising their execution.

"Second, by advancing the funds required for the execution of each approved project, all money thus advanced to be returned to the United States Treasury in a series of years upon completion of the project.

"Third, by waiving interest on money invested by the United States in such reclamation work.

"The fundamental principle should be that the cost of reclaiming lands and protecting property from damage by floods should fall upon the land reclaimed and upon the property protected in proportion to the benefits conferred.

"Action by the United States in any special case, if such a policy be adopted by Congress, pre-supposes request by the land owners for such action. No project should be carried out without a satisfactory pledge that the cost of the work will be returned to the United States within 20 years. Under such conditions as prevail in the Sacramento Valley, the reclamation work by irrigation might well be united with that of drainage, but the work there would be of such magnitude and should be carried forward with such energy that the utilization of the reclamation fund for this purpose is entirely out of the question. This fund has already been allotted practically to its limits.

"The attention of Congress might, therefore, with propriety at this time be called to the benefits which would result from a law substantially on the lines of the Reclamation Act, authorizing drainage projects to be examined and works for drainage to be constructed. Suitable provision should be made in such law for combining irrigation work with drainage so that comprehensive drainage and irrigation projects, similar to that for Sacramento Valley, would come within the scope of the law.

"It will then be for the property owners of Sacramento Valley to

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determine whether, all circumstances considered, they should rely upon the district system for an amelioration of the drainage conditions, or whether they should conform to the prescribed regulations and requirements and let the work be carried out by the United States Government.

"The Reclamation Fund, as above stated, is not available for work of this character, not even for preliminary examinations and surveys which must precede the approval of any project. In any law which Congress may pass to facilitate land reclamation by drainage, provision should, therefore, be made for setting apart a special fund to meet the cost of the reclamation works of approved projects.

"If such a law be enacted, it should authorize the Secretary of the Interior to make necessary examinations and surveys, and should place at his disposal and make immediately available not less than $200 000 for this purpose.

"If drainage works are to be carried out in Sacramento Valley on the lines now approved by State authority under co-operation of the United States, the State, and the drainage district, then at the proper time the problem of disposing of mining débris on some of the overflowed lands of the valley may properly be called to the attention of the engineers who pass upon the final plans of the required works.

"If, on the other hand, Federal authorities are put in sole charge of the work, then the examination of any comprehensive Sacramento Valley project would necessarily be extended to and would include all such features as the delivery of mining débris upon the basin lands of the valley."

Nothing came of the recommendation that a study of a comprehensive project be authorized.

There is no change in the situation to-day, except that active steps are being taken to improve the navigation and outfall conditions of the Sacramento River below Rio Vista. Rights of way for channel enlargement and channel dredging have been secured in whole or in part, and it is expected that to an appropriation of $400 000 by the State of California a like amount will be added by the United States Government, and that this money will be expended for the stated purpose under the supervision of the U. S. Army engineers.

This would be a first big step in the direction of improving the river's flood-carrying capacity, in line with the writer's repeated recommendations, notably as Consulting Engineer of the Commissioner of Public Works of California in 1895, and with the later project of the Commission of Engineers referred to by Mr. Foote and named in the foregoing quotation.

The problem of the Sacramento Valley is a large one, and cannot be dealt with adequately in a discussion of Mr. Foote's paper. The writer, therefore, will confine himself to very brief remarks.

River regulation problems are never exactly alike on any two river systems. The Sacramento is not in all respects like the Nile, though it does flow through a long narrow valley; the valley, however, is

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relatively much wider, and there are tributary streams, crossing first one side and then the other, each on an alluvial ridge, which complicate any such simple treatment as is so effective in the case of the Nile Valley. Neither could basin irrigation be relied on to produce such beneficial results as are hoped for by the author. The area of the flat submersible land in the valley is approximately 1 000 000 acres; nearly 2 000 000 acres of the valley lands are above the natural flood planes, and, if arranged as proposed for annual flooding, a very expensive system of large canals would be required to fill the marginal basins. When the broken character of the country in which such canals would have to be located is taken into account, it is foreseen that cost alone will be an obstacle not to be overcome.

Whether irrigated by flooding or not, there will ultimately be a system of low land basins in the valley. The main problem is one of drainage. Ample channel space must be provided for flood waters, and, when all the circumstances are considered, engineers will differ as to the best method of providing sufficient flood channels. The Board of Engineers referred to by Mr. Foote recommended (in 1905) channel enlargement, supplemented by a by-pass system of artificial channels. The proposed channels were inadequate in capacity, yet the Commission estimated that about $24 000 000 should be expended on the project, and it was fondly hoped by the land owners that the State and the United States would share in the cost, possibly in equal parts. Some years earlier—1894—while acting with Marsden Manson, M. Am. Soc. C. E., as Consulting Engineer to the Commissioner of Public Works of California, the writer had worked out a project for by-pass channels of much larger capacity, which, together with some channel enlargement, were to cost about $9 000 000. The smaller cost of the later project was due mainly to the fact that under the earlier estimates local drainage works were not included, and the only river work contemplated at the outset was in the lower stretches of the river. The writer very early formulated the following principles which he still believes will ultimately be followed in the treatment of the drainage problems of the Sacramento Valley:

1.—The river must be made to carry as much water as possible, ultimately perhaps all except the larger floods.

2.—The water which goes overbank must be let out at selected points, under control, so that it will not cut out large and deep crevasses.

3.—The water which leaves the river channel should not be allowed to accumulate in the valley basins, which are in part below the ordinary stages of the river, but should be carried in by-pass channels, under control, to an outfall.

The first requirement involves river enlargement, notably in the

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river's lower section, and particularly at Steamboat Slough, and also the construction and maintenance of an adequate levee system.

The second involves the preparation of long stretches of bank or levee for the overflow of flood waters during the flood stages.

Under the third requirement, a system of channels was proposed in 1895 between high levees. The largest of these by-pass channels was to be 4 500 ft. wide.

These matters are referred to in order that the magnitude of any drainage project for the Sacramento Valley may be better understood. Any such project is complicated, as Mr. Foote points out, by the irrigation requirements and by the mining débris problem. That irrigation is an essential requirement is not yet fully appreciated by the residents of the valley, but an awakening is apparent which is very gratifying. The mining problem is one requiring and deserving careful study and investigation, and it is receiving some attention at present in a small way. Dr. G. K. Gilbert, of the U. S. Geological Survey, is now making studies, at the University of California, of the power of flowing water to transport sands and gravels.

Mr. Parsons.

A. T. PARSONS, JUN. AM. SOC. C. E. (by letter).—There is a wide difference in the flood periods of the streams of the Great Valley of California. In the Sacramento Valley streams, extreme high water usually occurs in the winter and early spring, and the flow in May and June is nearly normal. As one goes south the comparative size of the winter floods decreases and that of the summer snow flood increases, until from Kings to Kern Rivers, inclusive, a winter or March freshet is usually of short duration and seldom gives as high a water mark as the big snow flood of May or June, which may stand near the top of the levees for two or three weeks. In this region, under Mr. Foote's system, it would seldom be possible to drain the basins before the end of June. This is late for putting in a crop, and, in addition, the weather by that time would be so hot as to crack the ground, and make it difficult to plow within a day or two after the water had run off. Mr. Foote's description of the Egyptian fellah sowing his grain before the water is entirely drained off, illustrates one of the many differences which would seem to exist between the problem of Egypt and that of California. In Egypt the crops are sown and harvested by cheap labor, in California by machinery operated by comparatively expensive labor, and the various devices for putting in a crop cannot be run over most California soils when they are very wet.

The author states that a channel to drain Tulare Lake would be expensive, but he hardly gives an adequate idea of how expensive it would be. The cut at the summit would be about 35 ft. and would exceed 30 ft. for about 10 miles. At present the main channel of Kings River reaches the trough of the Valley at a point close to the

summit cut proposed, so a great deal of work would be necessary on this channel also. As parts of Tulare Lake are as low as 185 ft., the proposed cut would have to be carried considerably beyond the 190-ft. contour. Mr. Parsons.

In general, the engineering difficulties of any comprehensive plan for reclamation in the Great Valley of California, great as they undoubtedly are, are small in comparison with the legal and financial problems involved. Any attempt at changing the flow through a "natural channel" is almost certain to result in a real or fancied interference with the vested rights of some riparian owner, followed by a lawsuit, and finally a permanent injunction. Few things impress a newcomer to the San Joaquin Valley so much as the light-hearted manner in which the old residents rush into water litigation.

Among other points of Mr. Foote's plan, his basin system would be almost certainly objected to by the owners of land below the higher basins, because of danger to their lands through the possibility of the upper dikes breaking; and, as Major Harts has pointed out, the adjustment of the cost of the project would be a matter of the greatest difficulty.

D. C. Henny, M. Am. Soc. C. E. (by letter).—A discussion of methods by which the present unsatisfactory conditions in the Sacramento Valley, as regards use and control of river water, can be improved, is of interest to any engineer who has given this subject thought. Mr. Henny.

The general subject of Sacramento River treatment has received much attention in the past. It was studied by the State Engineer more than twenty years ago, by the State Public Works Commission more than ten years ago, by a Commission of Engineers, generally referred to as the Dabney Commission, appointed by the State five years ago, by the new State Department of Engineering which was created two years ago, and by the United States Army Engineers. The river's use for irrigation and the possibility of storing flood waters has been investigated by the Reclamation Service Engineers; the stream flow, topography, and power possibilities have received attention from the U. S. Geological Survey in co-operation with the State, and the possibility of diverting or holding mining débris has been considered by the California Débris Commission.

The work of these several agencies has resulted in various reports and recommendations, each referring to one of the many problems presented. Commissions have been temporary in character and have had to base their conclusions on such scattered information as could be gathered, unsatisfactory to themselves as to continuity and completeness.

The author's paper is valuable because he sets forth clearly the

Mr. Henny.

great advantage of comprehensive treatment, and suggests a solution intended to cover the entire subject. Permanent spillways and the use of flood basins are advocated in general accordance with recommendations contained in the 1894 Report of the State Commissioner of Public Works, to which the author adds the feature of rapid drainage of basins for the better utilization, for agricultural purposes, of the flooded basins. He further endorses the construction, in connection with tributaries from the east, of débris barriers, and proposes the diversion of part of the water-carried material to adjoining low lands, and the construction of movable dams on the main stream in order to secure slack-water navigation.

Any project must depend for its feasibility on the details connected with design and cost. No details are furnished in the paper, which is admittedly suggestive, and, therefore, it can only be discussed as to general principles.

It is believed that Mr. Foote admits the disadvantage of flood relief through spillways, in silt-bearing streams, in view of probable shoaling, and that his project contemplates, as an essential item, the clarifying of the flood water by barriers, and diversion through under-sluices.

Much engineering skill and large funds have been expended in vain, in efforts to maintain a barrier in the Yuba River. The lesson thus far taught is that any such barrier is expensive to construct and difficult to maintain. A restraining dam which will permit floods to pass over safely can undoubtedly be built, but the introduction of artificial steps in the bed of a torrential stream imposes on future generations a very serious burden of maintenance, both by reason of the attack on the down-stream toe, which will be specially active until feasible storage is consumed, through the tendency of the cleared water to scour the river grades below, and later the heavy wear of detritus carried over, derived from natural erosion if not from old mining débris. When maintenance is fully considered, it may prove preferable—now that hydraulic mining is restricted—to cope with the accumulated débris, as far as it is not feasible to deflect it into side basins, by allowing it to find its natural course down stream, rather than to maintain it permanently in the stream bed itself in a condition of unstable equilibrium.

The application of the principle of under-sluicing to a torrential stream, in order to clear its water of part of its load, at times of flood, does not appear promising to the writer, because of the high velocities and the probable heavy cost. In any event, large quantities of fine débris, and ultimately of heavy detritus as well, will be carried over, and many tributaries must be left entirely unrestrained, for which reason it is not believed that any practicable barrier construction can be depended on to prevent injurious effect on sections in the river immediately below the over-bank spillways proposed by Mr. Foote.

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In Holland all by-passes for flood relief have been definitely closed during the last thirty years, in spite of the heavy expenditure involved in the enlargement of the lower reaches of the rivers to carry the full flood. The method there followed is to maintain a low-water channel suitably narrowed and defined by low spur dikes, and to place levees at proper distances to afford ample passage for the biggest floods. Whether this method is applicable to the Sacramento River situation, in view of the probable ultimate use of a large portion of the low-water flow for irrigation, remains to be determined. To the writer, however, it appears to be unsafe to hold that the concentration of even the biggest flood in the Sacramento River, within river levees, is, *a priori*, an engineering absurdity. It may prove to be excessively expensive, when the cost of the necessary land along the present river shores shall have been estimated, but this is by no means certain, nor is it as yet apparent that the enlarged and improved permanent river channel must necessarily coincide with the present river, placed as it is upon the top of a ridge and where adjoining land values are high, and that it cannot be shifted for long distances toward or to the troughs of the basins where land is cheap, in connection possibly with the canalization of abandoned river sections.

In regard to the comparative advantages of basin and perennial irrigation, the tendency in Egypt, as elsewhere, is toward the latter, even at heavy cost. Much of the fertilizing silt may thus be lost, but the advantage of control is attained, with the attendant greater certainty, variety, and value of crops.

The author proposes to retain full control, through suitable works, over that portion of the flood which the river channel will be unable to carry, and which will be by-passed. The river section at Sacramento has an area of less than 30 000 sq. ft., as compared with a maximum flood discharge into the Valley of more than 500 000 sec-ft., and a flow of 250 000 sec-ft. may have to be handled. The works necessary for the control of such quantities (with the slight grades available at many points) will undoubtedly prove a very expensive feature of the proposed project.

The writer fully concurs in the author's view, as implied in his scheme of movable gates for creating slack-water navigation, that the use of the summer flow of 5 000 sec-ft. in the Sacramento, at Red Bluff, and 2 000 sec-ft. in the eastern tributaries, for purposes of navigation, is wasteful, considering the necessities of irrigation; and that no scheme is likely to appear well balanced and satisfactory which does not provide for better navigation than is possible at present, and which will not accomplish this with a consumption of water representing but a fraction of the available summer flow.

The extent to which reservoirs in the mountains, if built primarily for irrigation, can be depended on to absorb a portion of the flood crest, is a mooted question, independent even of the lack of certainty

Mr. Henny. which exists as to the presence of suitable foundations for dams reported on and generally assumed as feasible. It cannot be doubted that some of these reservoirs will at times be nearly or entirely full well in advance of the flood reaching its maximum. Reservoiring constitutes one of the many branches of this intricate problem which must be given proper consideration.

To the writer it appears that, valuable as suggested solutions are, the subject can only be treated satisfactorily by comprehensive and sustained study. It may be advisable for this purpose to create some permanent organization, either under State auspices or under co-operative arrangement between the State and the Federal Government, with sufficient funds for continuous collection of information regarding the stream system, and for the systematic carrying on of a general study of the problem of river treatment, until a well-digested plan shall have been formulated. Certain work of river improvement, as now proposed, such as the enlargement of the river below Rio Vista, known to be necessary under any scheme, need not be delayed; other work, such as a change of the confluence of the Feather and Yuba Rivers, might be carried out by the State as an emergency relief for Marysville, and irrigation from Stony Creek by the Reclamation Service might be expanded as fast as funds become available. As soon, however, as a general scheme can be outlined and adopted, all further work should then be made to conform thereto. It will be possible, also, to work out a fiscal programme providing for the Nation or the State to be assessed on the land benefited. The control of rivers, to the extent that it does not belong to and is not exercised by the National Government, should be definitely assumed by the State, in order to prevent the further arbitrary contraction and obstruction of flood channels.

Mr. Foote. A. D. FOOTE, M. AM. SOC. C. E. (by letter).—In closing this discussion, the writer wishes first to thank Major Harts for calling attention to the error in regard to the inventor of the sector gate, and to offer a sincere apology to Colonel Chittenden for not giving him the well-deserved credit of it. The writer also wishes to express his appreciation of the valuable and generally helpful criticism of his paper.

There is, however, a certain amount of what might be called "destructive criticism," which shows either that the paper is extremely obscure, or was not carefully read. For instance, Major Harts states that "the author makes no provision for floods." As the entire paper is intended as a plan for providing for the floods of the Great Valley, there would seem to be no further discussion possible. Mr. Dillman evidently reads the paper differently. He says: "To capture the flood waters, hold them on the lands of the valley until they deposit their wealth of sediment, and then pass the decanted water on toward the

ocean, is certainly ideal." Both Major Harts and Mr. Dillman make a destructive and unnecessary criticism in the matter of sediment filling the rivers and waste channels, apparently forgetting the barriers which are to hold all sediment (as distinguished from silt) from coming into the lower part of the valley. If these barriers did not hold the sediment, it would settle in the basins and not reach the siphons and drainage channels. Mr. Foote.

Major Harts is very discouraging in the matter of building barriers, but the writer believes they can be built, and that Major Harts, with his former experience, would be the very man to succeed in building them. He would probably build the first one much higher than he did before, that it might form a pond large enough to store the first season's deposit and prevent the wearing caused by the coarse material running over the dam.

The feasibility of "squirting grout" needs no better proof than is furnished by Mr. Wadsworth's description of his experiments. With an adequate plant, which would have given him a pressure of 200 or 300 lb. per sq. in., it appears as if his success would have been complete.

The late Robert L. Harris, M. Am. Soc. C. E., a few years ago, made a long series of elaborate experiments, in which he showed that the process was eminently successful. He even made quicksand into solid stone, so that the process was afterward used in shaft sinking through quicksand and other water-bearing formations. Mr. Harris died soon after making his experiments, though he published some of his results, together with photographic illustrations.* The writer's experience confirms Mr. Harris' results, and he will continue to use the process, as he has before, where the conditions are right.

The feeling of uncertainty concerning the thoroughness of the spreading of the cement may be largely removed by leaving, at intervals, spaces a few feet square uncemented except for 2 or 3 ft. at the bottom to keep out the water. After the grouted portion has well set, the uncemented spaces may be dug out as wells, and the standing walls inspected. If there are uncemented spots they can be grouted, and the wells can then be filled again and grouted.

The arraignment of the Great Valley, in the first part of the paper, complained of by Mr. Wadsworth, should have been limited to the lands not under perennial irrigation, as were those which were compared with the lands of Egypt. With this limitation, the statements are substantially correct, even in the vicinity of Davis and Woodland, which he mentions.

William Hammond Hall, M. Am. Soc. C. E., in his able address to the Government engineers in 1905, gives a most authoritative and

* *Engineering News*, Vol. XXVII, p. 420.

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comprehensive description of the Sacramento River, from which the writer's statements in regard to its navigability were largely drawn, and they are not as misleading as the statement that there is as great a depth at low water now as there was in 1851. Mr. Hall states, and it is proved by the Government survey of 1907 (published in 1909), that in 1851 there were three or four bars only, giving 7 ft., with long stretches of deep and wide channels between. Now, there are almost continual, interlocking bars, giving that depth only in a very crooked, narrow channel, and, although the river commerce is considerable, it is carried on flat-bottomed steamboats and barges which struggle from one sand bar to another.

The policy of the Government, in guarding and controlling all navigation interests, is so well known that it was not intended or thought possible to put the navigation works under State control. The canalization of the rivers, which is practically what the plan contemplates, would, as a matter of course, be done by the Government engineers, working in harmony with the engineers of the State.

As Mr. Hall so well shows, in the address referred to, the Government is under a strong moral obligation to make those rivers navigable for high-class vessels, such as the canalization proposed would do. The Government should pay for all the improvements desired for navigation and the State should pay for the remainder. It is true that the movable dams are necessary for the State's interests, both in controlling the floods and in allowing all the water flowing in the dry season to be taken for perennial irrigation (except that needed for lockage). The feeding gates for the basins would be constructed in connection with these dams, which would be located along the river at such intervals as the Government engineers deemed best for slack-water navigation, and the costs would be apportioned according to the work done for navigation and for the State. The 250 gates and dams mentioned in Mr. Dillman's estimates, as well as the thousands on the side streams, are his own. The paper states that "one or two of these structures for regulation will be required on each of the smaller streams, and several on some of the larger ones."

Evidently, the paper is written so loosely that it has given rise to a misconception of the entire plan of basin irrigation. Except in a very slight degree, the basins are not for the storage of flood waters to relieve the rivers. The plan, as intended, is that the basins shall be filled as the muddy water arrives; when nearly full, the escape gates at the opposite ends of the basins will be opened into the drainage channels, and the muddy water will pass very slowly through them, depositing its silt. If the flood increases beyond the capacity of the drainage channels, the river dams will be lowered and all in excess will be passed down the rivers. These drainage channels are the by-passes of the Engineers' Commission, only they take their water from the

basins which are fed from the rivers. It is proposed that these drainage channels shall carry about one-third of the greatest floods, and that the rivers shall be reconstructed to carry the other two-thirds. Mr. Foote.

Neither does the paper seem to have made it plain that the whole scheme is based on catching the silt in the muddy water and spreading it over the land for the benefit of the tiller of the soil. All other objects contemplated are but by-products—the result of the works necessary for securing the silt. The vast sum of money required for the scheme is all contained in the silt, which will increase the value of the crop from $10 to $30 per acre, every year, on the 3 000 000 acres under the plan. It may be said that the low overflowed lands are fertile enough, and only need drainage, but they will steadily deteriorate under crops, and need the silt as much as any lands, if they are to be kept to their highest fertility. The argument of the paper implies that every gallon of water that goes to the Bay, carrying its load of silt, is so much waste of the fertility of the land. The ideal scheme would have the basins take every drop of muddy water through them and allow none to go down the rivers.

In this matter of the value of the silt there are no doubts among the tillers of the soil. Even the perennial irrigators are watering their lands in the winter for the silt the muddy water brings them. More than twenty years ago, Mr. James, Engineer of the Kern Land Company, at Bakersfield, reported nearly 20% better crops from land watered from the river than from Artesian wells.

The suggestion of Mr. Wadsworth to construct the drainage works first (including the rivers and the barriers to keep them free from sediment) is well made. The gradual construction and operation of the basins will have demonstrated their worth so well that, when they reach the margins of the perennial irrigation, the dovetailing or interlocking of the two systems will easily be accomplished. Mr. Veuve brings up the question very forcibly, and it is apparently a very difficult situation. As intimated in the paper, however, the basin irrigation (probably in a modified form) will be of vast benefit to perennial irrigation, both in fertilizing the land and in washing out the alkali. The writer believes that the Miller and Lux Company already feels the pinch of poverty and curse of alkali on its lands, and would gladly pay the price that would flood a portion of those lands each year, under a modified system, allowing, say, 2 ft. of water on the land for two or three days, which could then quickly be drained off, and be followed by a like flooding two weeks after; perhaps a third and fourth flooding might be obtained if the flood conditions permitted. The same would apply to the alkali lands below Fresno and other large areas in the San Joaquin. The flooding and quick drainage (the latter being the important part) will be the saving of thousands of acres of perennially irrigated land from alkali and poverty.

Mr. Foote.

In the matter of flooding houses and buildings on unirrigated lands, there are so few and they are so poor that they could be destroyed without material increase in the estimates. Where they are worth saving, they can be raised or diked off. The towns and cities, or any areas that would not be benefited by the basin system, would simply be left out of it. The river islands, which have been diked and reclaimed, most assuredly could not be taken in and be made to pay for their reclamation over again. It might be advisable for the State to take over the dikes, in order to have control of the entire works, but it would have to pay for them, either in cash or by some arrangement which would relieve the lands from all assessments.

The argument of Mr. Parsons in regard to the second or June flood is not well taken, as the winter flood will fill the basins and be drained off through the drainage channels in March. The June flood will be kept in the rivers, except that portion needed for the June irrigation of the basins, which will be considerable.

The question of plowing and sowing might safely be left to the ingenuity of the farmers and farm-machinery makers of the valley. It is suggested, however, that, as the perennial irrigation is shut off early in the fall, there will be water enough in the rivers to spread a light irrigation through all the basins, sufficient to moisten the land, and plowing could proceed for several months before the heavy rains came. After the basins are drained in March, and while still very wet and soft, a light "caterpillar" engine, with very wide "caterpillars" and a seed-sowing attachment, could be used, possibly with soaked seed, which, sprouting quickly, would very soon shade the ground. And here it may be mentioned that the farmer will not be pestered by vermin or rabbits, as all will have been drowned out; and the time for birds to steal the seed is very limited.

The objections to running the dikes across boundary fences and leaving a farm cut in two by a dike, with, perhaps, a small triangle of land on one side and the main portion on the other, will be overcome in a few years. The farmers will buy, sell, and "swap" their lands until the dikes, especially if they are built of concrete, will become the boundary fences themselves, to a great extent.

It is a matter of course that the people of the valley will have to agree to the scheme, or nothing can be done; but anyone who listened to the discussion of the Commonwealth Club of California, in the summer of 1909, on the swamp and overflowed lands of the valley, will agree that the "campaign of education" has advanced greatly. There is, very naturally, among the land owners, a great fear of entering into any scheme of improvement of which they cannot see and understand the outcome, which may tax them without limit, and also collect the tax before the work is done; but a proposition by which the State does the work, waits until the land can earn the tax, and then

collects it in extended and annual payments, is quite different, and the writer believes it possible to unite the people on such a proposition. Mr. Foote.

There seems to be some question as to whether or not the State can legally undertake the building of the works, or if it can, whether this will not entail endless litigation in collecting the assessments. As is often the case, to one who is not a lawyer, the law seems plain. The State can order the works built as public works of the State, and can issue bonds to pay for them. It can also create one large drainage district, to include the valley and mining counties, and, as the lands are benefited, collect a tax under the laws of the drainage district to pay the cost of the improvements and their maintenance; and this tax can be levied according to the benefits received. Land that now has a value of only \$2 or \$3 per acre (and that largely speculative), which will be made worth \$150 can be assessed \$50, while land now worth \$100 might pay \$10. A properly organized district government can work out all these details in an equitable and legal manner.

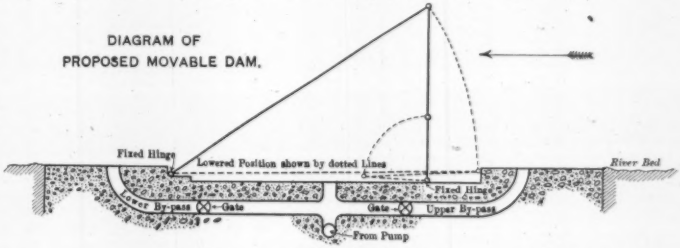


FIG. 5.

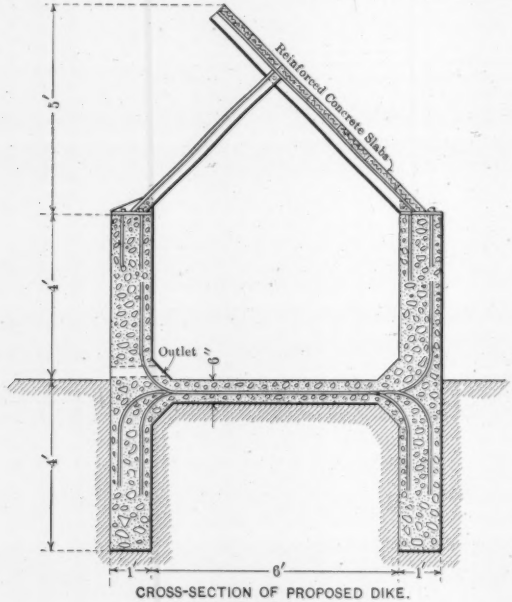
As an answer to the criticism that the foundations for the sector gate are expensive, the writer offers the rather dubious suggestion of building the gate to fold up, as shown in Fig. 5, and thus save the deep foundations. The pumps will lift the gate and hold it in all the lower intermediate positions, and the by-pass from above will control it for the upper positions and keep it up. Closing the upper by-pass and opening the lower one will lower the gate.

For the dikes built of earth, as described in the paper, it would probably be well in many cases to substitute structures of reinforced concrete and steel, somewhat as shown in Fig. 6, the saving in land occupied, and the absolute safety obtained, making up for the extra cost. The space between the walls would form a very good water-supply channel, both for flooding and for irrigation in June and September. A trenching machine would cut the trenches cheaply, and the foundations and brackets could be put in during the dry season, while the slabs, reinforced by some form of expanded metal, could be made under sheds during the rainy season when other work was slack, and hauled out and erected on fine days.

Mr. Foote.

It may be noted that concrete should be very cheap in the Great Valley, as there are large cement mills on its western edge and enormous stone-crushing plants near Folsom and Oroville, crushing the cobble-stones piled up by the gold dredges.

Of the interests in the valley: The swamp land owners will have their lands reclaimed. The wheat and hay lands will be enriched. Perennial irrigation will be increased by the use of all the water that flows in the dry season, as none will be needed for navigation; and



CROSS-SECTION OF PROPOSED DIKE.
FIG. 6.

many irrigators will be able to fertilize their land and wash away the alkali. The inhabitants of the cities and towns will be safe from floods, and will have an enormous increase in business. The miners will be able to "make wages" again; and the disgraceful conflict between the mountains and valley will cease. The splendid navigable channels will make transportation cheap. The railroads will have no more flood losses, and their freights will increase with the increase in production. San Francisco will take its toll from the annual $60 000 000 increase in the value of the valley products. The State will get the benefit of the reclamation of several hundred thousand acres of State lands, and will fulfill its contract with the Government

by which the swamp lands were secured. If there are any interests antagonized, they should be but a small minority. Mr. Foote.

In conclusion, the writer thinks the discussion has brought out difficulties and objections with great freedom and skill, yet it has not shown any serious obstacle to the success of the plan, unless it be the people of the valley. There are apparently no engineering difficulties that cannot easily be overcome, nor is there any doubt concerning the successful operation of the scheme when once completed, nor of the remarkably beneficial results that will follow. The writer can still say that the teachings of the 5 000 years of experience in Egypt lead to no experiment, but are a guaranty of success.

Summing up the foregoing, it seems to the writer that it would belie the intelligence of the dwellers in the Great Valley to say they cannot do this thing that was done in Egypt.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

TRANSACTIONS

Paper No. 1136

THE OUTLET CONTROL OF LITTLE BEAR VALLEY RESERVOIR.*

By F. E. Trask, M. Am. Soc. C. E.

The second largest development project in Southern California is that of the Arrowhead Reservoir and Power Company, on the headwaters of the Mojave River, on the north slope of the San Bernardino Mountains and about 13 miles north from San Bernardino, Cal. In this paper the writer will present only one of the minor features of this project, reserving the consideration of the dams, tunnels, conduits, and power plants for some future paper, when these works are nearer completion.

Little Bear Valley Impounding Reservoir is being formed in a mountain valley by the construction of an earthen dam at the narrows in the Little Bear Valley Creek. The water-shed above the dam has a drainage area of 6.62 sq. miles. The dam will have a height of 200 ft., and will impound 60 000 acre-ft., or its equivalent, 19 551 000 000 gal. The supply of water will be largely augmented from adjoining drainage areas, by diversion works at such elevations as to permit the delivery of flood waters through tunnels into Little Bear Valley Reservoir.

Outlet.—The general direction of flow of Little Bear Creek is eastward, the dam being at the eastern margin of the reservoir. The outlet is through a tunnel leading north through the mountains, and about

* Presented at the meeting of November 17th, 1909.

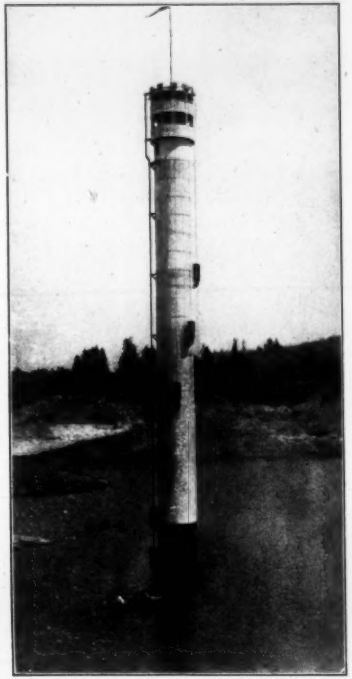


FIG. 1.—COMPLETED TOWER OF LITTLE BEAR VALLEY RESERVOIR.

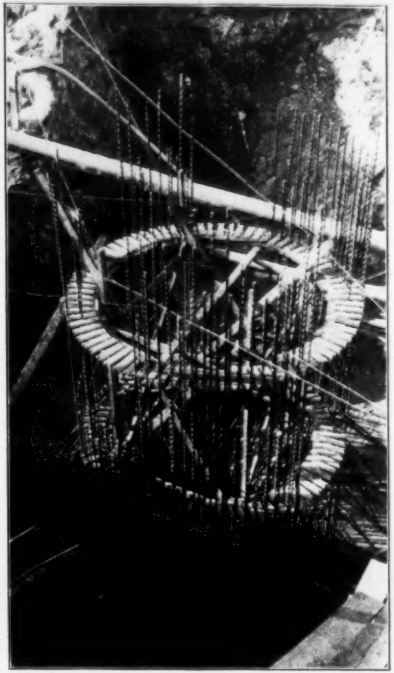


FIG. 2.—REINFORCING STEEL IN BASE AND FIRST SECTION OF TOWER.

FIG. 3.—METHOD OF HANDLING MATERIALS DURING CONSTRUCTION OF TOWER.

½ mile west of the dam. Fig. 1, Plate XXIII, shows the relative positions of the dam and the outlet tower.

Outlet Control.—The outlet tunnel plans and details of control were designed and partly completed some years prior to construction work on the present dam. Fig. 1 shows the original design for the regulation and control of the discharge waters from this reservoir. In 1906 it became necessary to complete the outlet works, and the engineering department, which, in the meantime, had been completely changed in its personnel in every branch, after much deliberation, concluded to finish the work substantially in accordance with this design, although it was regarded as a departure from standard practice and likely to prove a failure. This procedure was justified because of the fact that the cost of so doing was the installation of equipment already in stock and would add but little to the expenses of the final structure; and the further reason that this design could be subjected to one season's working conditions without delaying or jeopardizing the work on the dam.

The work was completed and the gates were closed during the winter of 1906-07. Before the close of the rainy season, in the spring of 1907, the water level had reached an elevation about 20 ft. above the tunnel portal, and as any further rise of the water would endanger the partly completed dam, the gates were opened sufficiently to maintain this stage of the water.

Trouble immediately began, from the clogging of the screens installed at the outlet portal. During storms, several men armed with grappling hooks were kept busy in boats over the portal, and it was with the greatest difficulty that the screens were kept sufficiently clean to maintain the water level below the danger point.

At the close of the rainy season the water in the reservoir was drawn down to the level of the outlet tunnel, and all accumulated drift was cleaned from the screens. An examination of the portal disclosed the fact that the metal-work of the screen chamber had been seriously bent and distorted, and that the masonry had been ruptured and partly destroyed. This one season's test demonstrated that a screened reservoir outlet chamber—which, with a full reservoir, would be submerged under 165 ft. of water and located in a lake 500 ft. from its nearest shore margin—was a dangerous innovation, not to be retained in any mountain flood-water storage reservoir.

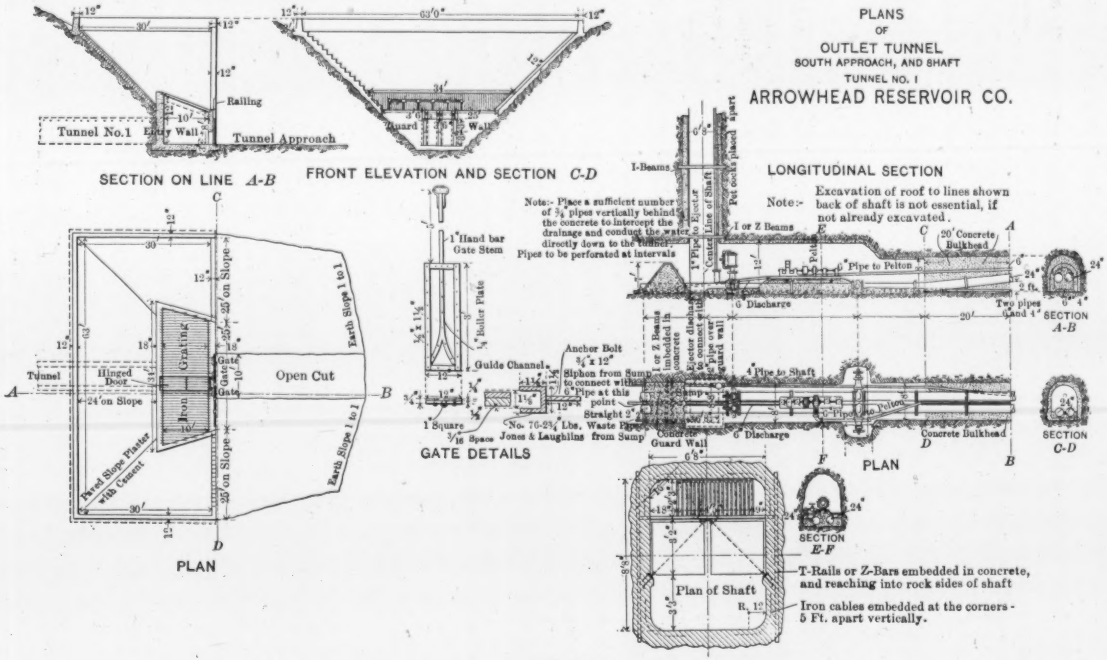

PLANS
OF
OUTLET TUNNEL
SOUTH APPROACH, AND SHAFT
TUNNEL NO. 1
ARROWHEAD RESERVOIR CO.
Tunnel No.1
Entry Wall
Tunnel Approach
Railing
SECTION ON LINE A-B
FRONT ELEVATION AND SECTION C-D
Guard
Wall
I-Beams
LONGITUDINAL SECTION
Note:- Excavation of roof to lines shown back of shaft is not essential, if not already excavated.
I or Z Beams
Pelton
Pipe to Pelton
6" Discharge
20' Concrete Bulkhead
Two pipes 6 and 4"
SECTION A-B
4 Pipe to Shaft
Concrete Bulkhead
SECTION C-D
PLAN
Note:- Place a sufficient number of 3/4" pipes vertically behind the concrete to intercept the drainage and conduct the water directly down to the tunnel. Pipes to be perforated at intervals
1" Hand bar Gate Stem
Roller Plate
Guide Channel
Anchor Bolt 3/4" x 12"
Siphon from Sump to connect with Pipe at this point
Straight 2"
No. 76-2 3/4 Lbs. Waste Pipe Jones & Laughlins from Sump
1" Square
3/16 Space
GATE DETAILS
Tunnel
Hinged Door
Grating
Gate
Open Cut
24 on Slope
Paved Slope Plaster with Cement
35 on Slope
Earth Slope 1 to 1
PLAN
Plan of Shaft
SECTION E-F
T-Rails or Z-Bars embedded in concrete, and reaching into rock sides of shaft
Iron cables embedded at the corners - 5 Ft. apart vertically.

FIG. 1.

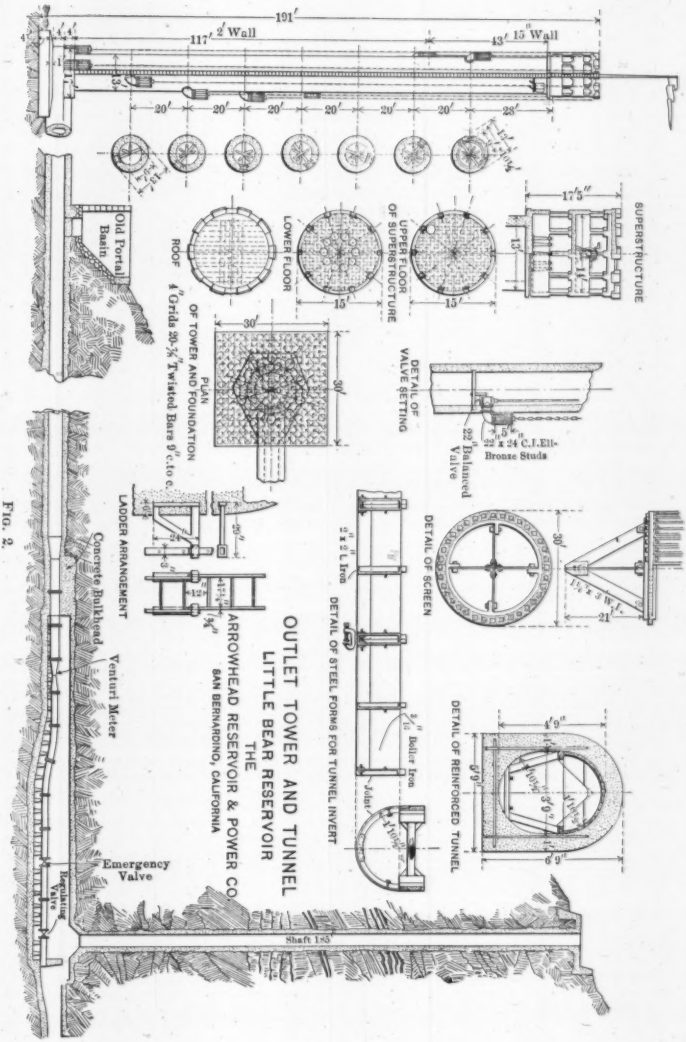

191′
117′ 2′ Wall
43′ 15″ Wall
20′
20′
20′
20′
20′
20′
28′
Old Portal Basin
ROOF
LOWER FLOOR
UPPER FLOOR OF SUPERSTRUCTURE
SUPERSTRUCTURE
17′5″
15′
15′
30′
30′
PLAN OF TOWER AND FOUNDATION
4″ Grids 20-⅞″ Twisted Bars 9″ c. to c.
DETAIL OF VALVE SETTING
22″ Balanced Valve
22″ x 24″ C.I. Ell-Bronze Studs
LADDER ARRANGEMENT
DETAIL OF SCREEN
DETAIL OF STEEL FORMS FOR TUNNEL INVERT
DETAIL OF REINFORCED TUNNEL
Concrete Bulkhead
Venturi Meter
Emergency Valve
Regulating Valve
Shaft 185′
OUTLET TOWER AND TUNNEL
LITTLE BEAR RESERVOIR
THE
ARROWHEAD RESERVOIR & POWER CO.
SAN BERNARDINO, CALIFORNIA

FIG. 2.

Reinforced Concrete Outlet Tower.—As a substitute for this condemned structure, it was decided to build a reinforced concrete screen, and gate-tower, near the portal of the tunnel, and connect it with the gate-shaft by a reinforced concrete conduit placed in the outlet tunnel. Fig. 2 shows the general features of the tower, conduit, tunnel, and gate-shaft, and includes the essential details of the tower and the conduit. Fig. 1, Plate XXIII, shows the tower completed and the water in the reservoir standing at a depth of about 20 ft. over the tunnel portal. The photograph, Fig. 2, Plate XXIII, was taken at the time the reinforcing steel for the base and the first section of the tower was set and ready for placing the concrete. Fig. 3, Plate XXIII, shows the tower about half completed, and illustrates the method of handling the material during construction.

A contract for the work was let, the company supplying cement, valves, castings, reinforcing steel, and all metal, and the contractor furnishing all labor, tools, rock, and sand, the contractor receiving and placing all metal parts.

The specifications required that the crushed rock should pass a screen having a 1-in. mesh and that the sand used should be from the creek bed or crushed from live rock and passed through a screen having a $\frac{3}{16}$-in. mesh. The relative percentages in the concrete mixture were subject to the regulation of the engineer, and as used gave the proportions: cement 1, sand 3, and rock 2. These proportions produced quite impermeable concrete when thoroughly spaded and tamped into the walls of the tower. Special attention was directed to bonding the joints between the work of succeeding days; the whole contact surface was chipped off and a groove 2 in. deep was cut on a radius 8 in. less than that of the outside of the tower. The whole surface then received a thorough washing, followed by a coat of cement grout, after which the next section of tower was placed.

The contractor carried on the work during the winter season, and provided against frost action by covering the top of the tower with canvas and maintaining a fire in the base during freezing weather. Under these conditions the progress was slow, as the concrete could be placed only when the weather was favorable.

The following figures are from the engineer's notes, and give the cost per cubic yard for the labor of placing 12½ cu. yd. of concrete in the tower at an elevation 70 ft. above the portal:

6 mixers	at	$2.50	$15.00
2 loaders	"	2.50	5.00
2 dumpers	"	2.62½	5.25
2 carpenters (tamping)	"	3.75	7.50
1 tamper	"	2.50	2.50
1 engineer	"	3.50	3.50
Engine rent			3.00
Fuel			2.50
Total cost			$44.25
Total cost per cubic yard			$3.58

These figures are much less than the average cost to the contractor, owing to the many delays and incidentals.

Design.—The problem of outlet regulation, presented to the engineers, required no new or radical departure from standard practice, and when the old work failed it was quickly decided to build a tower of reinforced concrete, with intake openings each 20 ft. in elevation, and equip each with a balanced valve and screen, the valves to be operated within the tower, from a house upon its top, the screens to be operated from the house, but outside of the tower.

The design further provided for connecting the tower with the outlet tunnel by a reinforced concrete conduit of such design and strength as to resist the crushing strains, when empty, resulting from a full reservoir with 160 ft. of water above the conduit. A gate control in the gate-shaft, in the original plans, will be retained, and the balanced valves in the tower will be used as emergency gates.

Sufficient material was put into this monolithic structure to give it gravity stability with a full reservoir, and the twisted steel rods incorporated were calculated to take all tension strains from the masonry, on a basis of 16 000 lb. per sq. in. of metal.

The resistance to overturning was computed on a basis of an empty reservoir and a wind velocity of 80 miles per hour. The formula adopted was for beams fixed at one end, with a uniform load; and the tower was analyzed for these and all other possible conditions and strains at each 10-ft. section from the base up.

The tower was built by Mr. A. S. Bent, of Los Angeles, a contractor of much experience in concrete work. Mr. Walter Hy. Brown,

Assistant Engineer (now Acting Chief Engineer), had charge of working out the details of the tower and of supervising its construction. The high class of work obtained is largely due to his untiring efforts and study.

The Consulting Engineers of the Arrowhead Reservoir and Power Company, Mr. F. C. Finkle, and the writer, of Los Angeles, Cal., are responsible for the general design of this structure, and have kept a general supervision over its construction.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1137

THE REINFORCED CONCRETE WHARF OF THE UNITED FRUIT COMPANY AT BOCAS DEL TORO, PANAMA.*

By T. Howard Barnes, M. Am. Soc. C. E.

With Discussion by John Hawkesworth, John B. Cameron, and T. Howard Barnes.

Among the several tropical divisions of the United Fruit Company, whose home office is in Boston, Mass., that in the Republic of Panama is of rapidly growing importance. It is called the "Bocas Division," from Bocas del Toro, the Provincial Capital, where the division headquarters are located, and is about 140 miles northwest from Colon. It may be noted in passing that the name is illogical, for its translation is "Mouths of the Bull." There are indeed several mouths from Almirante Bay and Chiriqui Lagoon, the bodies of land-locked water which form the magnificent harbors pertinent to this Capital, whence naturally the plural, "Bocas," is suggested, but the ascription of them all to the bull is an unwarranted monopoly on his part.

The fruit product at this port consists wholly of bananas which are planted on the suitable portions of the alluvial formation along the rivers entering the Caribbean Sea between Almirante Bay and Punta Mona, in Costa Rica.

In the past the method of collecting the fruit, in addition to picking it up by train from the farm loading platforms and its subse-

* Presented at the meeting of November 3d, 1909.

quent train transportation, involved its transfer to lighters having a capacity of from 600 to 1 200 bunches and then towing by launches, *via* river and canal, about 12 miles to the ship anchorage in Almirante Bay. Here the fruit had to be lifted by stages to the side ports of the vessels for its stowage.

It was desired to avoid the expense and damage involved in this lighter trip and the handling of the fruit, besides which there was damage by scorching and salt water spray. To this end it was necessary to build 13 miles of railroad through difficult country, several steel bridges (one, with its approaches, being ¼ mile long), and suitable terminal works. The latter included a railroad yard, a two-berth wharf with a trestle approach 575 ft. long, a merchandise warehouse, water supply, etc.

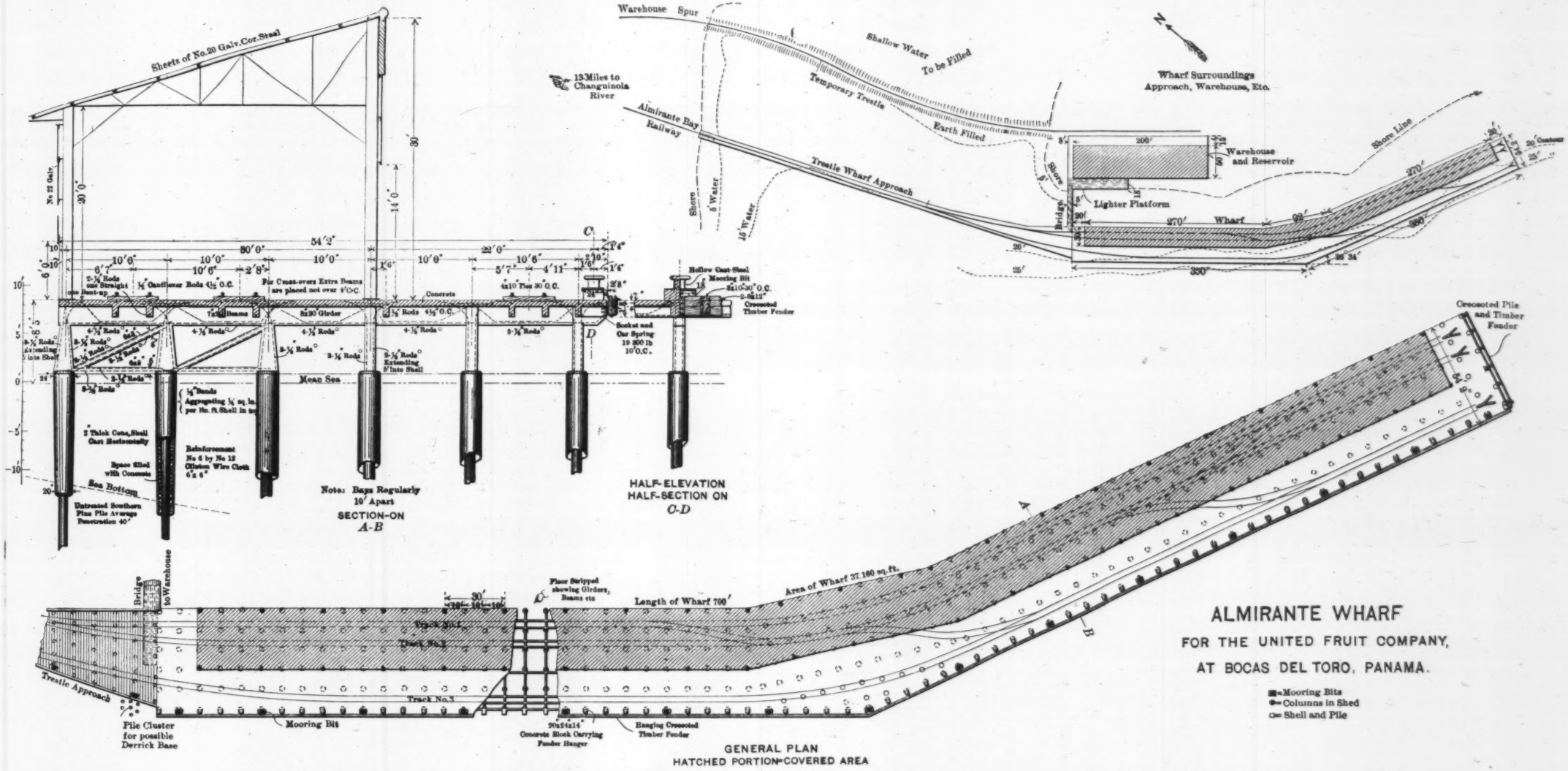
In September, 1906, the writer was engaged at Bocas del Toro in executing a contract for filling, sewering, and other sanitation work which the United Fruit Company had entered into with the Panama Government, and he was requested by the division manager to assume supervision of the construction of the wharf. Plans for a creosoted wooden structure were then under way at the home office, and it was expected to have it built under contract. At the same time the division manager further authorized the writer to prepare plans for a more permanent structure than the type proposed.

The *Teredo navalis* is the pest of wooden construction in those waters. So rapid is its work that ordinary native piles 10 in. in diameter are completely eaten through at the water line in a period as short as 8 months. Some varieties of wood last from 1 to 4 years, but these woods are scarce and of small size. The untreated southern pine quickly succumbs, it being noticeable that the teredo honeycombs the entire stick, including the resinous heart, leaving as the only outward marks the usual peppering of entrance holes perhaps $\frac{1}{30}$ in. in diameter.

As is well known, the activity of the teredo is for the most part confined to the vicinity of the water line, or between high and low water. As observed by the writer, it does some damage at depths as great as 20 ft., but none at all below the sea bottom. Creosote is fatal to the teredo, consequently its work on a treated pile proceeds only as the outer shell of the pile loses its creosote by swash or otherwise.

Along the Caribbean Coast of Central America the tide is slight,

PLATE XXIV.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXVI, No. 1137.
BARNES ON
REINFORCED CONCRETE WHARF AT BOCAS DEL TORO.



ALMIRANTE WHARF

FOR THE UNITED FRUIT COMPANY, AT BOCAS DEL TORO, PANAMA.

■=Mooring Bits
●=Columns in Shed
○=Shell and Pile

perhaps nowhere having a fluctuation of more than 15 in.; in Almirante Bay it averages 9 in.

There may perhaps be taken, as representative of the extremes in durability of creosoted piles on this coast, the experience at Port Limon, Costa Rica, which is little more than a roadstead, and Puerto Barrios, Guatemala, where there is a land-locked harbor. In the former port there is an almost constant ground swell which produces a violent swash. Some unprotected creosoted pine piles driven for fender purposes along the National Wharf were so badly eaten at the end of 5 years as to require replacing. The regular bearing piles here are copper-sheathed over felting, and have suffered not at all from the teredo, but have been badly eaten at points where the coppering did not extend into the sea bottom. In Puerto Barrios Harbor, where there is only a wind chop to produce a swash, creosoted piles have lasted 15 years before requiring renewal.

While it is true that in Almirante Bay no ground swell enters the part used as an anchorage, and much less the bight where the new wharf is built, nevertheless, a tenure of life of 15 years is short in view of the interruption to business caused by reconstruction; and, in respect to a copper protection, its expense would have added from 15 to 20% to the cost of creosoted construction, and coppering is not without constant expense for repairs. The writer immediately took up the question of a reinforced concrete type of construction, securing Joseph R. Worcester, M. Am. Soc. C. E., as Advisory Engineer. Various types of reinforced concrete piles were considered, but always with a hesitancy to adopt their use in view of the extreme length demanded, viz., 70 ft.

Finally, the writer made studies for using an untreated southern pine bearing pile, to be protected with pipes or shells of reinforced concrete. The shells were planned to be in lengths of about 4 ft., and having a diameter 6 in. greater than the outside of the pile. It was expected to use a diver in placing these pipes. The annular space was to be filled with lean concrete or even sand, embodying the steel reinforcement for the columns, at the pile heads, in richer concrete. The pile, thus enveloped and protected, was to constitute the base for the column and the imposed superstructure. Afterward, the plans were modified, and a single pipe or shell of tapering dimensions was adopted, as will be described later; otherwise, they were executed as outlined.

The estimated cost of a structure of this type was about the same as for a plain creosoted wooden pile and timber construction. The notable saving in favor of the reinforced concrete type lay in the smaller cost of the untreated, as compared with the creosoted pile.

When the new plans and estimates were presented to the home representative assigned to this division of the company, the decision to adopt that type was quickly made, and the manager was commissioned to execute the construction on company account.

Location of the Wharf.—The situation, shape, and immediate surroundings of the wharf are shown on Plate XXIV. The bottom near the shore is steep, so that the depth of the water is soon 50 ft. or more, a depth impracticable in which to build, and a condition which determined the location as shown, parallel with the contours, and affording berths on only one side of the wharf. It is to be observed that the depth of the water varies from about 10 ft. along the rear to about 22 ft. along the front of the wharf, affording a depth of about 27 ft. a ship's half breadth seaward therefrom.

Foundation.—The nature of the bottom was unknown, excepting by hand soundings. Accordingly, actual bearing tests were made on four 50-ft. piles driven in 9 ft. of water in permanent locations. A No. 3 Vulcan steam hammer was used on a floating driver. The total weight of the hammer and frame was 3 800 lb., the hammer proper weighing 1 800 lb. and having a throw of 30 in. About 130 blows per pile were used, the penetration of the last blows averaging $\frac{1}{2}$ in. The 3 800 lb. of dead weight sank the piles about 12 ft., leaving about 29 ft. of driving.

The sea bottom shows live coral on the surface, and all the indications are that coral in defunct forms extends as far as the piles penetrated. In line with this judgment it may be stated that existing shoals nearby (which have been excavated to a depth of 15 ft.) showed fingers of coral mixed with heads of the brain type.

The application of 11 tons per pile produced no settlement. A load of 16 tons produced a settlement of $\frac{1}{2}$ in., which did not increase with time, while 21 tons applied to each pile produced no further immediate settlement, but did produce one varying from $\frac{3}{4}$ in. to 3 in. in 4 days.

Load for Which to Provide.—The railroad locomotives constituted the chief concentrated load to be provided for. Owing to the easy grades

FIG. 1.—ALMIRANTE WHARF: SHELL RUNS, RECEIVING END.

FIG. 2.—ALMIRANTE WHARF: SHELL RUNS, DELIVERY END.

prevailing on the railroad, no locomotives exceeding 24 tons on the drivers of 11 ft. wheel base are required. With a spacing of 10 ft., a maximum concentrated live load of 6 tons per pile would be imposed. The dead load of concrete amounts to 9 tons per pile. A distributed live load much in excess of 6 tons per pile might occur at rare intervals, and the slab was calculated to sustain 250 lb. per sq. ft. During construction the wharf underwent an actual test closely approaching this, owing to the storage of some cement thereon. The capacity of the girders and beams to enlist through their rigidity the support of adjacent piles otherwise taking little live load may be depended on to increase the factor of safety. These considerations led to the neglect of a load of 3 tons imposed on each pile where a shed column should bear. The spacing adopted was 10 ft., and this was maintained as far as the shape of the wharf would permit.

Uses of the Wharf.—Plate XXIV shows the general construction of the wharf, the spacing of the piles, the arrangement of girders and beams, the location of the tracks, shed, etc. The design of the approach and tracks, shed, etc., is calculated to meet the demands for the prompt handling of fruit. Tracks 1 and 2 are for fruit, being under cover and furnished with ample cross-connections, whereby it is possible to keep a train of fruit constantly on one of the tracks at the berth in use, while cargo from the other berth may be taken off. At the present time, as no loading device has been adopted, and the passing of fruit by hand through the side ports is in practice, the outboard track (Track 3) is utilized for fruit loading, but machines will soon be used for carrying the fruit from Track 2 to the various decks of the vessel, and Track 3 will remain, as originally planned, to take incoming cargo for transfer to the warehouse.

General Description of the Wharf.—The framing system comprises bays of six columns capped with 8 by 30-in. girders, with longitudinal stiffening of 7 by 20-in. beams, one under each rail, as well as many others for cross-overs, unloading-machine bearings, and other stiffening. The dimensions of the wharf are shown on Plate XXIV, the length of each berth being 350 ft. on the front.

The three front rows of columns are 12 by 12 in. in section, while the rear rows are tapered from 8 by 24 in. at the top of the piles to 8 by 12 in. at the girders. The latter columns were thus shaped to grip the heads of the piles in a direction transverse to the wharf, and

constitute bracing piles by means of the struts and braces. A cross-section of the wharf and shed is also shown on Plate XXIV, and this gives a better idea of the bracing than can be obtained by written description.

The columns are 4 ft. 10 in. high, making the wharf floor 8 ft. 6 in. above mean water.

The floor slab is 7 in. thick. The thickness of the girders is increased to 20 in. at the front, and they are bracketed up from the front row of piles. In this thicker mass is moulded a cast-steel cup, designed to receive a standard car spring of 19 000 lb. resistance under a compression of 2 in.

Fender.—The front of the wharf is protected against friction and shock by an 8 by 24-in. bolted-up wooden fender. The fender is suspended from anchor-bolts, set in concrete blocks, by a link and hook-bolt passing through the fender, and rests against the spring. The contact is protected by a 12 by 12 by ½-in. plate set flush with the inner face of the fender.

Actual use demonstrates the efficiency of the fender, and that the shock, to be avoided as possibly disastrous, is to the ship and not to the wharf, for the floor acts as an immense horizontal beam which unites the resistance of all the piles, and, being supplemented by the existing dead weight, opposes the momentum of the ship. It is exceptional, in fact, for an impact to cause more than the closing of the spring in one or two sockets.

Unit Stresses.—The extreme fiber stress allowed in the concrete was 600 lb. per sq. in. in compression, and no account of tensile resistance was taken. An allowance of 16 000 lb. per sq. in. in tension was made for the steel reinforcement.

Reinforcement.—Round bars of mild steel of two sizes were used for reinforcement throughout: ⅞-in. for columns, girders, and beams, and ½-in. for floor slabs. Of the bars in girders and beams, one-half were cantilevered over the point of support. As before stated, the column reinforcement was planted in the annular space between the pile and the shell, and was sufficiently long to reach into the floor slab. No roundabout reinforcement of the column was used, because of its shortness; nor was any special reinforcement used in the girders and beams to meet shear strains, as the bent-up bars amply provided for this resistance.

FIG. 1.—SHELL PLACING, ALMIRANTE WHARF.

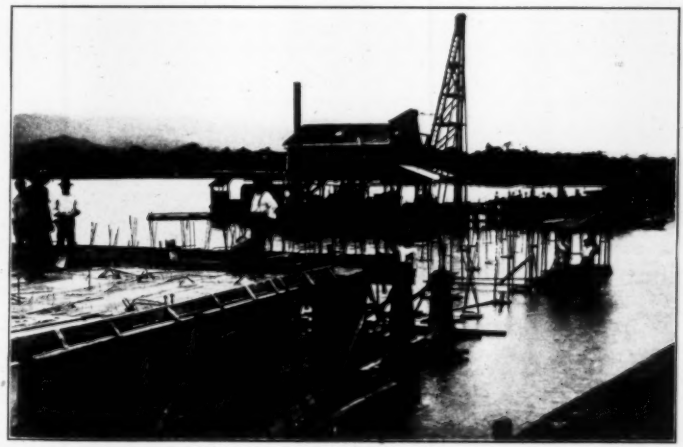


FIG. 2.—SHELL FILLING, FORM ERECTION, AND PLACING OF FLOOR CONCRETE, ALMIRANTE WHARF.

Considerable extra reinforcement was introduced into the 8 or 10-ft. strip of floor along the wharf front, in order to meet an unknown strain which a listing vessel might impose on the slab midway between the girders.

Anchor-bolts for the 4 by 10-in. by 5-ft. railroad ties were built into the slab, one for each tie, in staggered position.

The reinforcement of the shells was, for the most part, the electrically-welded fabric made by the Clinton Company. The mesh best adapted, and the one principally used, was 6 by 6-in. with No. 6 longitudinal and No. 8 roundabout wires. Some few home-made spirally-wound cages were made for shell reinforcement, but they proved to be expensive and were difficult to use.

Materials of Construction.

Piles.—Untreated piles of southern pine were used throughout the wharf proper. In the trestle approach, 575 ft. long, creosoted southern pine piles were used. The former were especially selected for their uniformity and the straightness of the butt.

Stone.—The stone used was a dense trap having a slight cleavage, but generally breaking crosswise thereof, and constituting a good ingredient. It was quarried on an island about 6 miles from the work, there loaded into small lighters, and towed to the site.

Sand.—The sand was brought on lighters from the Changuinola River. It was the best to be had, and, while very good, was not as sharp as one would select had there been other kinds from which to choose. For this reason, screenings consisting of fine stone and dust from the crusher were used freely, and much extra expense was incurred in re-crushing the larger stone for the production of this grade. In fact, no effort was spared to secure concrete of a superior quality.

Cement.—Alsen and Vulcanite brands of cement were used with entire satisfaction. Field tests only, from actual mixtures, were made. The total quantity of cement used was 3 450 bbl.

Concrete.—An average mixture of 1:2:4 of cement, sand, and stone was used, the sand usually containing some crusher dust and fine stone, which would make the mixture really $1:1\frac{1}{2}:4\frac{1}{2}$. The idea of producing a dense concrete was kept foremost, especially in making the shells, on which any destructive effect of sea water would be mani-

fest. The ingredients were varied to suit the requirements for particular portions of the structure, being made especially fine and rich in the columns and in the bottoms of girders, beams, and slabs.

From time to time, 4-in. sample cubes were taken from the concrete as mixed. Four of these cubes were tested by the Worcester, Mass., Polytechnic Institute, with the co-operation of Chester S. Allen, Jun. Am. Soc. C. E., who was Assistant Engineer on the construction. The following are the results, the samples being 15 months old when tested: 2 367, 4 911, 3 355, and 5 367 lb. per sq. in., an average of 4 000 lb. per sq. in.

Details of Construction.

Plant.—The plant available in the division consisted of a No. 2 portable Climax crusher, an engine, an elevator, and bins. These were utilized, although they were not what would have been selected as first choice. The working base was an island shoal, the soil of which was very yielding, thus causing much annoyance and expense. In spite of 2 ft. of filling which had been pumped on the area, settlement took place, so that high tides covered much of the ground. The crusher engine was removed from its position on the boiler and set on a pile and concrete base. A rope drive from the engine actuated the No. 2 Ransome concrete mixer and hoist.

A 28 by 75-ft. hull was used for the floating pile-driver. The latter was equipped with leads 56 ft. high and the Vulcan No. 3 hammer previously described. A land driver was also brought into requisition for some driving for the warehouse and other structures on shore.

Shell Forms.—The 4-ft. shells, or rather pipes, which were at first planned to be used for pile protection, were never made, but, as stated previously, single shells long enough to reach from above high water to below the sea bottom were substituted, and these were reinforced so as to stand handling and placing by the pile-driver.

Soundings were taken at each pile location, and the shells were made of sufficient length to permit their sealing into the sea bottom and to reach above ordinary high water. Furthermore, the shells were made and stacked in the order in which they were to be used, thus greatly facilitating their placing.

The attempts to utilize adjustable forms and home-wound wire

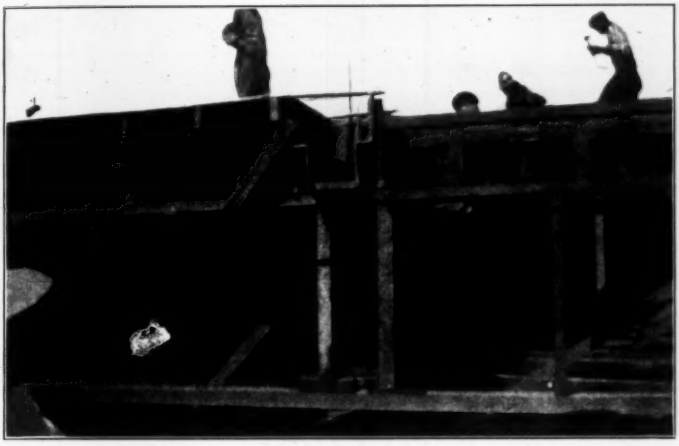


FIG. 1.—FRONT VIEW, ALMIRANTE WHARF, SHOWING SHORING, ETC.

FIG. 2.—REAR VIEW, ALMIRANTE WHARF, SHOWING FINISH.

reinforcement need not be described; but it should be noted that such forms must be very firm and stiff, and capable of preserving the desired thickness of shell, qualities which adjustable forms do not possess; and that it is too difficult to adjust any closed wire cage over the core. The forms adopted leave improvements to be made. They consisted of five parts: two half cylinders of No. 16 gauge galvanized iron, a spider or spacer of wood, a wooden cradle lined with sheet metal, and a wooden arch faced with sheet metal.

The half cylinders—really not cylinders but frustums of a cone—when fixed over the spider formed the core of the shell. They tapered from 20 in. to 16 in. in diameter, whatever the actual length might be. The cradle rested horizontally on a truck, and, together with its companion arch, formed an outer mould having a diameter 4 in. greater than the corresponding core, thus giving a thickness of shell of 2 in.

The exact length of shell desired was secured by stops, in the form of collars, made of short sections of wood attached to sheet metal. They served the further purpose of keeping the core concentric with the outer form. The forms were made up in lengths of 16, 18, 20, 24, 28, 30, and 32 ft.; there were several of each length, excepting of the longest ones.

Shell Making.—The stone ingredient of the concrete was necessarily limited to a size not exceeding ¾ in. in diameter. By careful grading, a mixture yielding 13 cu. ft. of concrete to the barrel of cement was obtained. The order of making was first to place the proper thickness of concrete along the bottom of the cradle, and therein embed the reinforcement. The latter opened longitudinally, and was wired together when the next step was taken, that of setting the core in place. After closing the wire about the core and clamping down the latter, the concrete was filled in to the "springing line" of what constituted the arch of the shell as it lay in a horizontal position. The arch form was then clamped on, and the remaining concrete was placed, openings along the top of the arch form having been left for this purpose. The concrete was compacted by churning it into place with light flat iron bars and by tapping the outer moulds. This method was effective in expelling the entrained air, and made a very solid concrete. Where in contact with the forms, the concrete was left very smooth, and the only additional finish was a wash of grout.

The open spaces at the top were necessarily left to be finished by hand. The forms were slightly oiled to assist in the removal of the shell.

The freshly moulded shells were carefully rolled along the track a minimum distance, and allowed to stand over night.

Shell Curing.—While the concreting was going on, two other gangs were at work, in addition to those mixing and placing the concrete. One small gang was preparing the cores and placing the stops, and a larger gang, equal to that required at the moulding, was busy unloading and stacking the shells. The concrete set very rapidly, so that all shells excepting those 25 ft. or more in length could be unloaded on the storage runs the morning after they were made. The spider was drawn first, and then the core forms, the upper one of the latter requiring ordinarily tedious cutting away of the fine rich concrete which had oozed into the lap of the upper over the lower portion of this form. Then the top or arch form was removed, and the cradle was rocked on its side, thus throwing the shell out on the run. Here it was inspected and patched, the latter being required in but few instances, and was kept moist for several days (Providence greatly assisting). The runs consisted of two lines of supports along the shells at their quarter points. A slight inclination toward the water facilitated the movement of the shells on the runs.

As stated, most of the shells were less than 24 hours old when they were unloaded; the longer ones were a day older, yet so rapidly does concrete set in an air temperature ranging from 75° to 90° in the full 24 hours, that the total losses from all causes amounted to only 2 per cent. Each shell was marked with a letter and number indicating its position.

The description of this process of producing the shells sounds simple in its recital, and so indeed it is, but the forms were heavy and the preparation was somewhat tedious; the men needed constant watching and spurring on to their task, and the result was that, with delays occasioned by using the moulding force to hurry up the preparation of the forms, or to unload some of the shells, it was possible to produce regularly not more than six shells per day. Altogether, the production of the shells was the hardest portion of the entire work.

Shell Placing.—The placing of the shells proved to be very easy. A special box punt, decked over, was used for the conveyance of the shells. This was tied broadside at the foot of the run, and the shell

FIG. 1.—FRONT OF ALMIRANTE WHARF, BEFORE FIXING FENDER.

FIG. 2.—FRONT OF ALMIRANTE WHARF, SHOWING FENDER HANGING.

was rolled thereon over short skids. The punt was towed by rowboat to the pile-driver, and the shell was lifted by the fall with a rope sling into vertical position and placed about the pile just driven. The shell was allowed to drop with a slight momentum in order to bed the toe, and any further settling needed was secured by resting the hammer and frame on the shell top, or by slight drops of the hammer, with a cushion of rope and plank interposed. In some cases the shell was not long enough, indicating a settlement of the coral about the pile. In these cases, a sheet-metal casing was clamped about the head of the shell, and, usually with some difficulty, the head was extended to the proper grade. The time consumed in placing the shell was about six-tenths of that used in driving the pile.

Shell Filling.—After wedging the shell firmly in a concentric position about the pile head, a sufficient quantity of fine, rich concrete was deposited quietly within to seal about 2 ft. in depth at the foot of the shell. After two days or more it was generally possible to pump out the enclosed water, and then, under dry conditions, the lean concrete could be placed. This mixture was carried to within 5 ft. of the top, where the column reinforcement was embedded. The concrete for this portion was made of the rich mixture. Then the pile with its concrete envelope was jacked and clamped into position ready for the forms of the superstructure.

Concrete Forms.—The forms were made of ⅞ by 4-in. matched lumber, with battens disposed for clamping by yokes and wedges. The shoring of the floor hangers (joists) and beam bottoms was carried on a system of railroad iron resting on the shell tops, with beams laid crosswise on the rails. The entire erection, with its yoking and shoring, demanded conscientious attention, even old hands not always realizing the necessity for inspecting carefully each and every point of bearing, while, as for novices, one needed the search-light eyes of the crab to supplement their delinquencies.

The raising of a system of centering as involved as that required to shape short columns and 6 by 8-in. braces is a tedious operation at best, but, when conducted over the water, fortitude and patience are prime essentials. The forms were used repeatedly, and in most respects served their purpose admirably. The forms provided were sufficient to center 110 lin. ft. of the wharf, entailing their use six times or more. They were given a light coat of oil at each erection after the first.

Placing the Concrete.—The mixture in the shells and other portions was placed quite wet, which was necessary in order to fill properly the spaces in areas of small cross-section like those dealt with. The method of churning and tapping proved most effective in the columns, braces, girders, and beams, in which portions the depth of the concrete assisted in its consolidation.

Constant vigilance was required to secure proper results. The Jamaican negro seems to think that, when he has once been through any operation properly, "that settles it, it's done forevermore." A smooth but not a polished surface was given the floor slab; this was done by local masons, but they too required like supervision in order to secure proper results.

The columns were generally moulded 24 hours in advance of the girders and beams; and the period between the latter and the floor slab was seldom less. The placing of the floor slab concurrently with the girders and beams produced a shrinkage crack along the plane of the bottom of the slab. An interval as short as 6 hours was at times allowed, but, as a rule, a longer period was maintained, although the defect mentioned did not appear after 6 hours' setting.

The work of the day on beams and on floor slabs was cut off in the centers of the spans between girders, generally a strip 20 ft. wide across the wharf being carried at one operation.

Very little patching was found necessary when the forms were stripped. The 6 by 8-in. braces were the members most requiring this attention. The tops of the shells, with the contained concrete, were rounded neatly by hand about the foot of the columns, and the entire concrete surface was coated with thick cement grout applied with brushes. Several persons (laymen) paid the compliment of inquiring what kind of wood was used for veneering.

All the concrete was conveyed in Ransome buggies from the hoist delivery, involving a maximum carriage of 400 ft.

Steel Assembling.—The system of reinforcement being that of "loose rods," much care was required in setting the steel and maintaining it in place. In comparison with the vigilance required in placing the concrete, this exaction was not severe.

The rods were shaped with a simple hand-bending lever worked on a bench, the angles desired being chalked on the top thereof. The cost per pound for cutting, bending and assembling was $0.0028. The

FIG. 1.—LOOKING EAST ALONG ALMIRANTE WHARF.

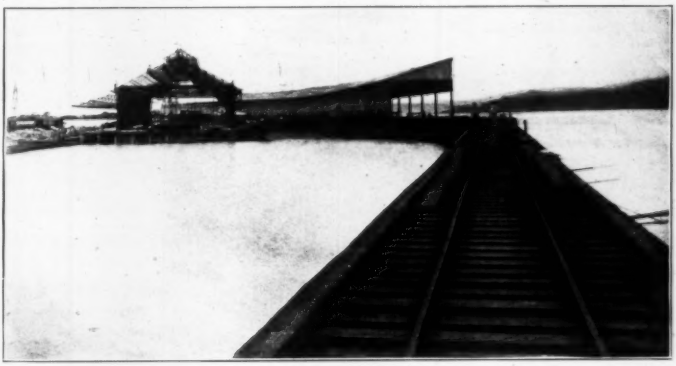


FIG. 2.—ALMIRANTE WHARF: LOOKING EAST FROM TRESTLE APPROACH, SHOWING WAREHOUSE, SHED, ETC.

total weight of the reinforcement was 250 200 lb., exclusive of that in the shells, which was 2 700 lb., or 6¾ lb. per sq. ft. of wharf area, and amounted to 1½% of the volume of the concrete calculated above the level of the shell tops. The steel was furnished by the Baltimore Bridge Company.

Superstructure.—The shed over the fruit tracks extends the full length of the wharf, less 20 ft. at each end. The width is 30 ft. from center to center of the columns, the latter having the same spacing longitudinally. The frame is of steel, and was roofed with No. 20 galvanized-iron sheets; portions of the sides were covered with No. 22 sheets of the same material, as indicated on Plate XXIV, which also shows the shape of the structure.

The weight of the structural work was 308 000 lb. The cost for distributing, straightening, and erection was $2 300, or $14.91 per ton of 2 000 lb. The paint and painting (two coats) cost $991, or $6.44 per ton. The labor on the covering, including the louvers, cost $939, exclusive of the painting.

Warehouse.—Of interest allied to that of the wharf are some of the points of construction involved in the warehouse and water supply. The warehouse is 50 by 200 ft. on the column lines. In addition to this, there is a platform 14 ft. wide along the service railroad, over which there is a structural steel roof extending to the center of this track. There is also a platform 9 ft. wide at the west end of the warehouse, and from this extends a bridge connecting with the wharf and serving for pedestrian and light trucking uses.

The warehouse foundation is on piles. A mass of concrete with roundabout reinforcement was used to form a head and base on each pile. The grade of the floor is 6 ft. above mean water, and the available space between that and the natural surface (Elevation 0) was utilized for a reservoir by building a floor, finished at Elevation 1, and a walled enclosure. The reservoir is divided into two compartments which may be used alternately or in combination, the total capacity being 300 000 gal.

The present supply for this reservoir is the roof-water from the warehouse, with the possibility of admitting at any time that from the wharf shed. The water is conducted about the yard in pipes of liberal size, and will serve as the potable supply. The necessity for pumping

the water is expected to prevent waste. The inner surfaces of the reservoir were treated with a soap and alum wash.

In commending the appearance of this structure, the writer need make no apology, as it was built mainly during his absence. To the credit of the superintendent and others contributing, it must be said that the integrity and finish of the concrete is second to none.

Water Supply.—A supply of water under pressure, for boiler and laundry uses, was obtained from a creek 2 miles inland. A small storage reservoir was provided on a hill, near the point of taking, and about 100 ft. above sea level. This reservoir is about 200 ft. from the line of the main leading to the wharf. It is of reinforced concrete, and is circular in form, having 6-in. walls, and is covered with a dome 4 in. thick. Its internal diameter is 30 ft., and it is 12 ft. deep, affording a storage of substantially 60 000 gal.

The interior surfaces were treated with a wash of soap and alum. There was no seepage when the reservoir was filled, which speaks well for the integrity of the work.

The main is of galvanized wrought-iron, having a diameter of 5 in. from the pump to the reservoir, and of 4 in. from the reservoir branch to the wharf, along which it is continued, with a branch to the warehouse. There are abundant outlets for service to ships as well as for the terminal structures, railroad yard, etc. Whenever the ordinary consumption becomes a serious factor in producing friction in the main, a small local storage capacity at the terminal will be of advantage.

Statistics.—

First pile driven, August 16th, 1907.
Last pile driven, March 27th, 1908.
First concrete poured, November 15th, 1907.
Floor finished, April 3d, 1908.
Area of the wharf floor = 37 160 sq. ft.
Number of piles in wharf = 432.
Number of shells in wharf = 432.
Average length of piles = 58 ft.
Average length of shells = 18.4 ft.
Quantity of concrete in shells = 7 230 cu. ft.
Quantity of concrete in filling shells = 6 180 cu. ft.
Quantity of concrete in columns, etc., = 35 100 cu. ft.

FIG. 1.—REAR VIEW, ALMIRANTE WHARF, LOOKING WEST.

FIG. 2.—REINFORCED CONCRETE RESERVOIR AND PIPE LINE.

Cost of wharf floor	$79 377
Cost of approach	15 836
Cost of bridge to warehouse	608
Cost of fender	2 273
Cost of shed	26 770
Cost per square foot for:	
Labor for mixing and placing concrete	$0.08
Labor in making, erection, and removal of forms	0.204
Materials for 6 500 sq. ft. of forms	0.454
Same into area of wharf	0.08
Wharf floor, exclusive of shed	2.13
Cost per linear foot of shell	1.78
Cost of average shell	33.40
Cost per square foot of shed, structural work	0.94
Total cost per square foot of shed	1.38
Labor cost per 1 000 ft. of lumber on approach	25.90

The cost of general items, in which are included engineering, superintendence, watchmen, cooks, storage, and general supplies, was 24.2% of the other expenses, and is included in the costs above given.

General.

Labor.—The labor was almost entirely recruited from Jamaican negroes resident in the vicinity. There were also some Central American Spaniards and some Patois, but almost all were of African blood, or had a considerable mixture thereof. The Spaniards had the least African blood, and were the most intelligent, while the Patois were the least desirable, with the Jamaicans close competitors. In indolence, both the latter are hard to outclass, being also slow-moving to an exasperating degree. It is necessary to bear in mind constantly this limitation, apparently incurable, in order not to lose one's nervous balance.

The regular wages were $1.00 gold for 9 hours' work increasing to a maximum of $1.50, which was paid to riveters, so rated, but not experts. The average was about $1.20. In point of service rendered, the $1.00 man was worth say 40% of the value of an ordinary Italian (not an experienced one), which would make the latter figure $2.50 per day.

Quality of Work.—Credit must be given to the General Superintendent, Mr. Robert V. O'Brien, and to the Assistant Engineer, Mr. Chester S. Allen, for securing a quality of concrete, both in integrity and external finish, very gratifying to the writer. The foregoing remarks about the class of labor available will suffice to explain the difficulties under which the work was accomplished. Mr. Alphonse Briffod was in charge of the pile-driving, and Mr. F. Herbert Mallett was foreman of form making, erection, and removal, and ably seconded the efforts to achieve these results.

Remarks on Forms.—The writer would emphasize the value of regularity in forms. In the statement of costs that of 8 cents per sq. ft. for the labor in mixing and placing concrete may be compared with 20.4 cents per sq. ft. for the labor in making, erecting, and removing the forms. This is a significant contrast. In the case in hand there existed a section 100 ft. in length, comprising the central and widest portion of the wharf, where regularity could not be maintained. The irregularity in the positions of the railroad beams also served to retard the rapid erection of the forms. These beams might have been used on regular spacings had it been practicable to give the floor slab sufficient strength to take the concentrated locomotive load in whatsoever point. In two designs which the writer has in hand at present, it is possible to design the slab in this way with economy of materials, and without imposing too great a dead weight.

In conclusion, a word of caution should be given: In construction of this kind care should be taken to extend the protective shell to a depth secure against any future erosion and consequent exposure of the wood. Recent examples of the destruction of even creosoted piles by sea life at depths of 25 ft. emphasize this caution.

DISCUSSION.

JOHN HAWKESWORTH, ASSOC. M. AM. SOC. C. E. (by letter).—The writer has examined this paper with much interest. From the author's statements it is quite clear that, under the conditions, unprotected timber piles could not be used. The scheme of casting protective shells of concrete appears to have afforded a practical solution of the problem. However, it would be interesting to know whether any investigations were made as to the comparative cost and practicability of regular reinforced concrete piles, the greater cost, of course, being offset in part by the smaller number required. Mr. Hawkesworth.

If the concrete shells should develop fine cracks, from temperature stresses, wave abrasion, or the impact of vessels against the wharf, what opportunity would be afforded to the teredo? If the hole made by the teredo is only $\frac{1}{10}$ in. in diameter, as stated, it would not take much of a crack to facilitate his entrance to the wood inside.

In regard to the reinforced concrete design, it appears that the slab reinforcements run between beams and parallel to the 8 by 30-in. girders, the span varying from about $3\frac{1}{2}$ to 10 ft. Although it is stated that the slab was designed for 250 lb. per sq. ft., the amount of reinforcement appears to be just as great for the short as for the long slab spans, and the entire slab load appears to be carried by the beams, which are primarily designed as stringers to carry the tracks. As the span between girders is also never more than 10 ft., it would seem that a more rigid and economical design might have been obtained by running the slab reinforcing bars in both directions, and increasing the spacing of the bars.

The author makes no statement as to the distance allowed between reinforcing bars, and the under surface of beams, girders, or slabs, nor as to the assumption in regard to the width of slab considered as the flange of the T-beams for girders and beams. This renders it difficult to check the strength of the design or its suitability for 24-ton locomotives, but the statement that no shearing reinforcements were needed, the bars being simply cantilevered over the points of support, would seem to be open to criticism.

JOHN B. CAMERON, ASSOC. M. AM. SOC. C. E. (by letter).—The writer has read this paper with especial interest as it brings to mind the ravages of the teredo on wooden dock construction in Alaskan waters. Mr. Cameron.

In 1903-1904 the Alaska Central Railway built a dock, using native spruce piling, and although it seemed to be free from the teredo for some time, it was completely destroyed within 18 months.

It having been stated that cottonwood piling would not be infested by the teredo, considerable quantities of this wood were used when the dock was re-driven. The teredo attacked the cottonwood, however,

Mr. Cameron.

but not until long after the spruce was destroyed. Evidently, the teredo will not enter cottonwood through the bark, and only attacks it at places where the bark has been removed. The bark, being tough, did not peel much under the hammer, and resisted decay for some time after driving.

The opposite was true of spruce, which was almost completely barked by the operations of logging and driving.

The writer cannot state positively that the teredo will not enter cottonwood through the bark, but careful investigation of infested piles led to that conclusion, which the writer believes to be a fact.

This dock was destroyed within 6 months after the second driving, the spruce piling being cut off and dropping out. It was re-driven with creosoted fir from Puget Sound. At the time, several plans for more permanent construction were considered, but the first cost was much in favor of creosoted fir.

The form of construction described by Mr. Barnes would be favorable for docks in Alaska, as local supplies of piling and gravel can be found there.

Mr. Barnes.

T. Howard Barnes, M. Am. Soc. C. E. (by letter).—The writer notes the very pertinent queries in Mr. Hawkesworth's discussion. As stated quite briefly in the paper, reinforced concrete piles were first considered, one of the reasons for not using them being the extreme length required, 70 ft. At that time, piles of that length had not been successfully produced, and the inadequate yard room available served to exaggerate the difficulties of casting, storing, and other handling.

A wider spacing, which Mr. Hawkesworth suggests as possible with concrete piles, was not admissible in this case. The additional dead and live load per pile would have called for still greater length, already too great, as before stated.

In regard to the opening up of fine cracks in the concrete shells, thus admitting the teredo, the causes for producing such cracks cited by Mr. Hawkesworth, namely, temperature stresses, wave abrasion, and impact of vessels, do not operate in this locality. The water temperature ranges between 80° and 85° Fahr.; there are no waves, only a light wind chop; and, as to impact, the vessels buff on the fender, without possibility of listing far enough to touch the shells. It may be noted further that the habits of the teredo cast great doubt on its operating in a sort of *cul-de-sac,* which such a crack would constitute, the water being probably too dead to satisfy its craving for air.

In general, the resistance which such an envelope offers against the forces mentioned depends on the adequacy of the reinforcement and the character of the concrete. The impact of vessels should be avoided in any case, for their own protection, as much as, or more than, for the protection of the wharf members. The reinforcement used in the Bocas wharf shells is not as heavy as that demanded for

Mr. Barnes.

extreme temperatures with freezing conditions. The character of the concrete should be perfect as to density and integrity. This was possible under the system of casting used, as it afforded full opportunity for inspection and curing, and the writer believes such conditions were fulfilled in the work described.

The slab reinforcing bars were carried from girder to girder, with additional cantilever bars in cross direction laid over the track beams. In Plate XXIV, the dots representing the section of the slab bars were reproduced as dashes, hence the criticism justly applied by Mr. Hawkesworth.

No width of slab, in addition to that of the girders and beams, was considered in the calculations. The moment was taken to be that due to a distributed load of two-thirds of the locomotive weight (24 tons) over the two track beams, 7 by 20 in., spanning between the girders, 10 ft. from center to center.

The writer desires to acknowledge the suggestion of this type of pile protection, by Mr. F. A. Koetitz, in a paper before the Technical Society of the Pacific Coast, in April, 1906. He has recently learned that, since this date, this form of protection has been patented by Mr. Koetitz, who has kindly favored the writer with some photographs of work, embodying this protective feature, executed by the Pacific Construction Company, in San Francisco, of which Mr. Koetitz is Vice-President. The writer regrets that he did not have the benefit of that company's experience when the Bocas wharf was being built, for the production of the shells was the most difficult part of the work.

The writer also regrets that some discussion of the stability of concrete in sea-water has not been brought out by this paper. He feels indebted to the counsel of William B. Fuller, M. Am. Soc. C. E., that density be attained as the greatest protection against the action of sea-water, and is pleased to record that close observation of the shells at the date of writing (January, 1910) reveals no outward sign of deterioration of the concrete after the lapse of 27 months. Observations on other concrete structures in the tropics, which have shown some disintegration under the action of sea waves, emphasize the importance and efficacy of density in such locations.

It is greatly to be desired that additional information be obtained on this most important feature in the use of concrete in sea water.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1138

CHARACTERISTICS OF MODERN HYDRAULIC TURBINES.*

By Chester W. Larner, Esq.

With Discussion by Messrs. Lewis F. Moody, John C. Parker, C. M. Allen, E. Kuichling, Daniel W. Mead, S. J. Zowski, and Chester W. Larner.

The design and selection of turbines for hydraulic power-plants were at one time—and that within comparatively recent years—processes of an extremely uncertain nature. They partook quite largely of experimentation, much of which, having no sound basis of scientific knowledge, degenerated into the purest guesswork. During late years, however, the various problems involved have been the subject of much careful and intelligent study, and the situation has become decidedly better, although there is still room for vast improvement.

One of the chief obstacles in the path of progress at the present time is the scarcity of published data on the performance of turbines of recent construction. A large quantity of exceedingly valuable information has been accumulated by the manufacturers, but very little of it is available to the public in published form, or, in fact, in any form whatever. The publication of tests seems to be an idea altogether abhorrent to the majority of turbine builders in America. It is with this policy of reticence that the writer is inclined to take issue, not only because it is of an unprofessional character, but because

* Presented at the meeting of October 20th, 1909.

it defeats the very commercial ends which, according to some peculiar method of mistaken reasoning, it is supposed to further. It is only by a broad-minded policy of publicity on the part of the manufacturers, that the Engineering Profession at large can become acquainted with the possibilities of turbine construction, and thereby profit in the selection of this apparatus by the progress which is being made in the development of the art.

The publication of data relative to results which have been accomplished does not necessitate the publication of any information concerning the details of design on which these results depend. Any manufacturer is justified in protecting for his own use any points of superiority in design which he may be able to devise. His success depends on them, and he could not reasonably be expected to publish them for the benefit of his competitors; but the results which can be accomplished by his machines in operation are quite a different matter. This information is of vital importance to all engineers interested in the development of water-power, and it is in pursuance of this conviction that the writer has prepared this paper.

In the evolution of turbine design in America, probably no factor has played a more important part than the Holyoke testing flume. Up to the present time, more than 1 800 turbines have been tested there, and these tests have undoubtedly had a very important influence on the advancement of the art. The fact that all wheels (within certain limits of capacity)* are tested there under essentially equivalent

* The Holyoke flume is best adapted to the testing of wheels from 27 to 42 in. in diameter, approximately. At the time the flume was built, wheels were of much lower capacity, and it was not designed to handle the large volume of water discharged by high-speed wheels 45 in. in diameter or greater. The writer names 42 in. as the limit, not because he believes that it is impossible to get good results with a wheel of larger size, but because no engineer of judgment would think of installing a larger wheel to operate under similar conditions, and dependence cannot be placed on such tests. If the conditions are less favorable than would be found in an ordinary plant, they are certainly unsuitable for a test where the object is to determine the best performance of which the wheel is capable. The main defect in design which affects the tests of large wheels is the shallowness of the tail-race. The floor of the flume, on which the draft-tube of the wheel stands for test, is only 5 ft. above the flat bottom of the tail-race. The draft-tube of a 49 or 50-in. wheel, unless it is very short, will be about 7 ft. in diameter at the lower end. The back pressure due to a column of water 7 ft. in diameter impinging on the flat bottom of the tail-race, 5 ft. from the end of the draft-tube, can be easily imagined. It has a throttling effect upon the discharge which cuts down the effective head, but does not show on the gauge, and hence the efficiency, as shown by the test, suffers. The loss does not seem to be altogether proportionate to the quantity discharged, but, apparently, is affected very largely by the relation of the draft-tube diameter to the depth of the tail-race in which the column has to turn to flow toward the weir. A 7-ft. column of water cannot readily be turned through an angle of 90° in a space of 5 ft. without considerable disturbance. The writer on one occasion tested a wheel which discharged 275 cu. ft. per sec. and found that the tail-water level, as determined by pressure readings from the draft-tube, was actually 1 ft. higher than the level in the well where the head gauge floats. The head, as shown by the gauge, was about 10 ft., whereas the real head on the wheel was only about 9 ft. Naturally, it is useless to expect good efficiency under such conditions. This is an extreme case, but wheels of smaller size, down to about 42 in. in diameter, undoubtedly suffer in the same way, although to a less extent.

conditions, adds greatly to the value of the results, because it renders them directly comparable. Tests made in place are subject to the effects of so many local conditions that one scarcely knows when it is fair to make direct comparisons, but Holyoke tests are not subject to this uncertainty, because the conditions are known and constant.

Further than this, the writer believes that it is not going too far to state that the Holyoke tests are more accurate than a majority of those made elsewhere, and are as accurate as the best of them. There is no other device for measuring power which is as generally reliable as the friction brake, when accurately adjusted. At the Holyoke flume the power of all wheels is determined by friction brakes, of which there are several sizes. The water is measured, according to the Francis formula, by a sharp-crested weir of adjustable width. The width of the weir is adjusted according to the discharge of the wheel, in order to bring the head on the weir within the limits of the experiments on which the Francis formula is based. This is in accordance with the best practice elsewhere. Furthermore, the arrangements for taking simultaneous readings, and the system of testing in general, are much superior to the conditions which often obtain in the case of tests in place.

These facts, together with the unquestionable impartiality of the tests, have given them a position of generally accepted reliability. Certain imputations to the contrary have occasionally emanated from Europe, but these are best answered by the fact that, in not a few cases, European engineers who have transplanted their activities to America have failed conspicuously to attain in the Holyoke flume the good results with which they were accredited at home.

Holyoke Tests versus Tests in Operation.—It should not be thought, from the foregoing remarks, that the writer regards a Holyoke test as necessarily an exact determination of the duty which a turbine will perform after installation. It is scarcely ever that, although, by the exercise of proper judgment, it may be a reasonably reliable indication of what may be expected in regular operation.

Tests are of interest to the Engineering Profession at large for two reasons: first, in order to determine what results have been accomplished by a machine which has been built and installed; second, as evidence of what a builder has accomplished in the past, on which to base speculations as to what he can accomplish in the case of a

proposed installation. In the first case, a test in place is generally regarded as the most satisfactory, but, in the second case, the writer finds a great diversity of opinion regarding the relative merits of Holyoke tests and tests of units in regular operation. It is conceded by all, of course, that an accurate test in operation is the only exact method of determining the performance of the turbine under working conditions, and it is further conceded by most engineers that the tests which they themselves make are accurate. However, they are not so credulous in regard to the tests made by others. The apparatus used for tests at the site is usually improvised for the occasion, is handled, to a certain extent, by unpracticed hands, and the opportunities for error are many. For this reason some engineers place more confidence in Holyoke tests because they know that they are correct and that they furnish a reliable basis on which to estimate the results which may be expected after installation.

On the other hand, some assert that wheels are tested at Holyoke under ideal conditions which cannot be obtained in practice; that the test shows merely the efficiency of the runner, and a lot of other generalities, none of which has in general any sound foundation of fact. There are certain conditions under which a Holyoke test is more favorable to the wheel than a test in place, and there are other conditions under which it is decidedly the opposite. As a matter of fact, the results of any test, whether made at Holyoke or at the site, cannot be relied on to indicate the performance of a prospective installation under different conditions of head, or speed, or capacity, without very careful study of the influence which these changed conditions will have on the efficiency of the wheel. For example, to infer that, because a manufacturer has obtained certain efficiencies with a wheel developing 10 000 h.p. under 150 ft. head at 200 rev. per min., he can obtain the same results with a wheel developing 10 000 h.p. under 100 ft. head at 300 rev. per min. would be entirely unwarranted and erroneous. Such comparisons, however, are often made, sometimes to the disadvantage of the builder, and frequently to the delusion of the purchaser.

In order to elucidate this matter, the factors which affect a comparison between wheels operating under different conditions will be analyzed. For this purpose the losses of energy in the turbine may be divided as follows:

(*a*)—Hydraulic loss in the casing, from the penstock flange to the entrance to the guides,

(*b*)—Hydraulic loss in the guides and runner,

(*c*)—Hydraulic loss due to leakage around the runner,

(*d*)—Hydraulic loss in the draft-tube,

(*e*)—Mechanical loss due to the friction of the revolving parts.

In respect to loss (*a*), the advantage is usually in favor of the Holyoke test. It is ordinarily impracticable to test a runner at Holyoke in the casing in which it is to be permanently installed, and this fact must be taken into consideration. As installed in the flume, the wheel draws the water to the guide openings smoothly and with minimum disturbance, from a state of comparative rest. Naturally, the efficiency is higher under those conditions than it will be when the wheel is installed in a usually all too constricted casing in which the water approaches the guides with whirls and eddies due to obstructions and restricted areas. There is no form of casing for a turbine that can be made within the limits of reasonable proportions, except the spiral, which does not cause considerable loss of efficiency. The loss in a properly designed spiral casing is relatively small, because it accelerates the velocity gradually and delivers the water to the guides without abrupt change of direction. A test of a wheel in an open flume can be fairly applied to a turbine of the spiral-casing type with very little allowance for additional loss, but a considerable deduction should be made if the wheel is to be installed in a cylindrical casing.

Loss (*a*) is dependent largely on the velocity in the casing, and is related to the head only in that for a given velocity the percentage of loss decreases as the head increases.

Loss (*b*) is independent of the head, provided the speed is allowed to vary as $\sqrt{h}$, but is materially affected by the type of runner and the design of the guides. The results obtained with one type of runner cannot be assumed to apply to another type. This point is fully treated in the paper.

Loss (*c*) is likewise independent of the head, and is as great at Holyoke as in any other case. Both the leakage around the runner and the discharge through it vary as the $\sqrt{h}$, and hence the ratio of one to the other (or the percentage of loss) does not vary at all.

Loss (*d*), in the case of small wheels, is probably as low at Holyoke as elsewhere. With large wheels, however, this loss is excessive.*

Loss (*e*) varies with the head, decreasing as the latter increases, because the power of the wheel increases very much faster than the friction loss due to the weight of the revolving parts. There seems to be a general impression abroad that in testing wheels at Holyoke the mechanical friction is so far minimized as to be practically eliminated. It is true that effort is made to reduce the friction load, but it is not eliminated, by any means, and, although the percentage of loss is less than it would be for the same wheel running in horizontal bearings under the same head, it is more, and sometimes a great deal more, than it would be for the same wheel operating in a horizontal position under a higher head. In fact, it is necessary to minimize the friction in order to show results which will be representative of what the wheel can accomplish under heads of from 30 to 50 or 60 ft. For wheels to operate under higher heads, a Holyoke test is, almost without exception, a disadvantage to the wheel. This fact can be readily demonstrated. Consider, for example, one of the wheels referred to later, Test No. 1799. This wheel developed 112 h.p. under 17.25 ft. head. Suppose it is desired to install a pair of these wheels on a horizontal shaft under 80 ft. head. Will the efficiency be more or less than it was in the testing flume? Under this heading, of course, the influence of mechanical friction only will be considered.

This wheel when tested had four vertical guide-bearings, one ball thrust-bearing, and a stuffing-box, in contact with the shaft. The weight on the ball-bearing, including the unbalanced thrust of the runner, was about 7 000 lb. The pressure on the upper guide-bearing under the brake was 1 000 lb., due to the pull of the load. Under 80 ft. head, two of these wheels would develop

$$\frac{112 \times 2 \times \sqrt{80^3}}{\sqrt{17.25^3}} = 2\,240 \text{ h.p.},$$

which is twenty times as much power as was developed under test. The weight of the revolving parts would be about 5 000 lb.

Now, making the impossibly liberal assumption that under test the friction of the three lower guide-bearings, the stuffing-box, and the ball-bearing was absolutely nothing, there still remains the upper guide-

* See foot-note on p. 307.

bearing, which is known, beyond question, to carry a load of 1 000 lb. Since the bearings of the proposed unit will carry a weight of 5 000 lb., the friction will be about five times as great (neglecting the influence of speed, which would modify this statement slightly).

The power developed, however, is twenty times as great, and hence the percentage of energy lost is only one-quarter as much as when the wheel was tested at Holyoke. If the friction under test was 2%, then the friction in operation will be only 0.5%, or a difference of 1.5% in favor of the latter. Under yet higher heads, the wheel in operation has a continually increasing advantage from this source.

In spite of these facts, the writer has known engineers of experience to make the broad statement that friction is so completely eliminated from Holyoke tests that they may be considered to represent purely ideal results which cannot be obtained in practice, and this in reference to heads of even 200 or 300 ft. Such statements will not stand under analysis. Under high heads, exactly the reverse is true. In fact, the writer knows of no case so ideal or so favorable to high efficiency as that of a large reaction unit operating under a high head. In such a case, not only is the mechanical friction reduced to a minimum effect, but the casing usually takes the spiral form, which minimizes the casing loss. The wheel usually has individual draft-tubes, which minimize the draft-tube loss, and the design of the runner approaches the purely inward-flow Francis type, the characteristic of which is high efficiency. The design of the wheel throughout is the most efficient which can be devised.

Scope of Tests.—Tests of turbines after installation are usually made for the purpose of determining whether a specific set of conditions has been complied with satisfactorily. These tests are usually limited to observations at various gate openings under the head and speed for which the turbine was designed. Where the power is measured electrically, the difficulty of determining the efficiency of the generator at varying speeds precludes the possibility of a comprehensive test. Such tests may fulfil their function, which is to determine the performance of a guaranty, but they add very little to the knowledge of turbine characteristics in general.

It is customary practice to design a turbine to perform a certain duty under a certain head, and either to test it under that head, or else under whatever head is available, and, from the data thus

obtained, compute the performance of the wheel under the head for which it was designed. Excepting high heads, there are few installations, however, where the conditions are by any means constant. A test made at constant speed under one head is of little value in determining the performance of the wheel at the same speed under another head. It is of even less value in determining the general characteristics of the wheel, and, as will be demonstrated later, these characteristics are of vital importance in considering its adaptability to a given service. The characteristic curves of a wheel cannot be plotted from a test at constant speed. It is essential that the speed should vary through a considerable range. In this respect, the Holyoke tests are excellent, and, but for the fact that they are so seldom published, they would furnish a vast amount of useful information.

Selection of Wheels.—The writer is of the opinion that more trouble has resulted from the misapplication of turbines to conditions for which they are not suited, than from any other of the many mistakes which have been made in the equipment of water-power plants. There are numerous instances all over the country where a wheel which would give good results if run at the proper speed is being run at some other speed and giving correspondingly poor results. The cause of these mistakes is not far to seek. It lies chiefly in the failure to analyze the tests of these wheels and thus determine whether they are properly adapted to the existing conditions.

Comparatively few engineers realize to what an extent the performances of different types of turbines differ. The divergence is wide, and the speed selected determines largely the type of runner which must be used. In spite of this, the speed is usually determined from a consideration of generator economy alone, without regard to its effect on the economy of the turbine. Such a course is short-sighted and ill-considered, to say the least. It results in high-speed wheels being adopted where lower-speed wheels are more suitable for many reasons. Frequently, much care and ingenuity are devoted to shackling the manufacturer with rigid guaranties for apparatus which should never be used at all for the purpose intended. Premiums are even paid to secure results which, economically considered, would be expensive if obtained gratuitously.

There is a more and more insistent demand for high-speed wheels, without a proper consideration of the limited conditions to which they

are adapted and the undesirable attributes which all of them possess. The high-speed wheel, as will be explained later, has its legitimate field, but in many plants where it is being used to-day it is by no means the most economical type.

Nor has the manufacturer failed of his share of responsibility in the misuse of his wheels. Because of the long-established practice in America of standardizing runners and building lines of stock sizes, there has been too much effort directed toward adapting the plant to the wheels, rather than the wheels to the plant. This unfortunate practice has been due largely to the "cut and try" methods of the McCormick school of turbine designers, which, until recent years, dominated the turbine business in America. Each builder, having by tedious experimentation evolved his standard patterns, was accustomed to select from his stock the wheel which, according to his judgment, would most closely approximate the desired results. Frequently, the approximation was a very remote one, not more, probably, because of the rigidity of his standards than because of his meager knowledge of their characteristics. It is only within the last few years that manufacturers have learned to analyze the tests of their wheels intelligently, and to predict with accuracy what they will do under a variety of conditions. It is still more recently that designing of special wheels to suit individual requirements has come into general practice.

The writer believes that the special wheel is to be the rule in the future, rather than the exception. This should not be taken to mean that every new wheel will necessarily be an entirely new and original design. Such a procedure would be of too experimental a nature, particularly in the case of mixed-flow wheels, which are not as completely amenable to mathematical analysis as are their more simple prototypes, the Francis wheels. New designs will consist rather of modifications of and improvements on wheels which have been tested and found to be efficient, and, as such, will be more certain of success than if based wholly on theoretical considerations. The important point to be observed is that the wheel should be exactly suited to the conditions, in order that the best results may be obtained. There should be no approximation.

Description of Tests.—The purpose of this paper is not to present a study of the various problems which arise in the selection of water-wheels, but to illustrate, by means of tests, the individual charac-

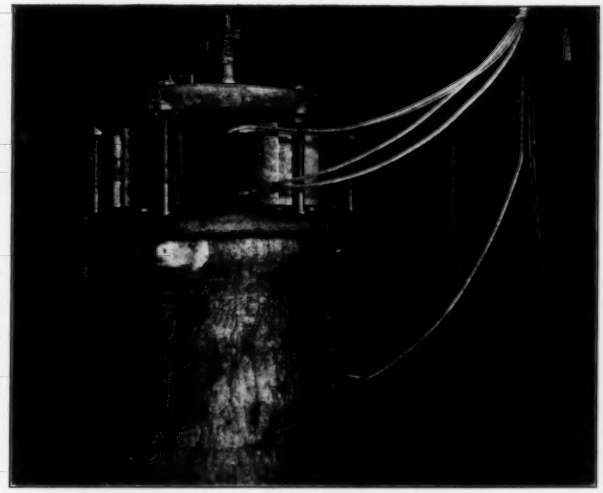


FIG. 1.—WHEEL IN FLUME READY FOR A TEST.

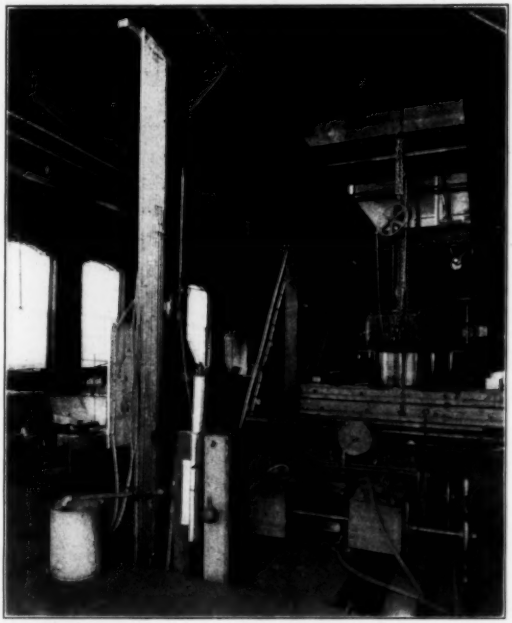


FIG. 2.—OPERATING FLOOR, SHOWING BRAKE, GAUGE-BOARD, ETC.

teristics of several different types, and to indicate in a general way the conditions of service to which each is best adapted. The conclusions drawn from this study are based, not merely on a consideration of these tests alone, but also of all the other American and European tests which have been accessible.

The tests which have been used for illustration were made recently at the Holyoke testing flume on a series of five specially designed runners. They are reproduced from the original reports in Tables 1 to 5, inclusive. These runners range from the low-capacity Francis type wheel, No. 1795, adapted particularly to high heads, to the large-capacity high-speed wheels, Nos. 1796 and 1797, adapted particularly to low heads. The average maximum efficiency of the five wheels is 88.06 per cent. This is believed to represent the best results thus far attained in the practice of the art. Tests Nos. 1799 and 1800 are especially notable in that they show higher efficiencies than any other reliable published records in either American or European practice. Test No. 1799, showing an efficiency of 90.43% at three-quarters-gate, is probably very close to the maximum obtainable economy.

The five wheels were successively tested on a vertical shaft in the same casing. The gates were of the balanced-wicket type. All the runners were of cast iron, made in one piece from cores.

Fig. 1 is a sectional view of the arrangement for testing one of the larger wheels. The vertical thrust was carried on a ball-bearing placed on top of the crown-plate. This arrangement was necessitated by the variation in height of the runners. It was not adopted to minimize the mechanical friction, and it is not believed that the friction was any less in the case of these tests than is usual with wheels of the same size. The usual arrangement is to carry the thrust on lignum vitæ step-bearings, but the invariable practice of all manufacturers is to turn off the face of the block until a small button about 2 in. in diameter is left for bearing. While running on this button the friction is practically nothing, although it takes more of a pull to start than is required with a ball-bearing.

Fig. 2 shows the arrangement for wheel No. 1795, with an interior wooden draft-tube. This inner tube was not used with any of the other wheels.

Fig. 1, Plate XXXI, is a photograph of the complete apparatus erected in the flume ready for a test. This view shows an arrangement

TABLE 1.—TESTS OF A 32-INCH R. H. WELLMAN-SEAVER-MORGAN COMPANY TURBINE WHEEL, No. 1795.

Date, February 18th and 19th, 1909. Case No. 1794.

Wheel supported by ball-bearing step. Swing-gate. Conical draft-tube.

Number of experiment.	Proportional Part of: Percentage of full opening of speed-gate.	Proportional Part of: Percentage of full discharge of wheel.	Head acting on wheel, in feet.	Duration of experiment, in minutes.	Revolutions of wheel per minute.	Quantity of water discharged by wheel, in cubic feet per second.	Horse-power developed by wheel.	Percentage of efficiency of wheel.
69	1.000	1.024	17.23	4	112.50	34.45	46.84	69.59
67	1.000	1.009	17.50	3	152.67	34.22	55.80	82.16
68	1.000	1.008	17.57	2	163.00	34.25	57.31	83.98
66	1.000	1.005	17.46	3	164.00	34.05	56.90	84.40
65	1.000	0.992	17.47	3	172.67	33.60	55.92	84.00
64	1.000	0.979	17.48	3	181.33	33.17	54.53	82.93
63	1.000	0.965	17.46	4	189.25	32.67	52.53	81.21
62	1.000	0.946	17.47	4	213.00	32.06	49.27	77.57
61	1.000	0.688	17.59	4	251.50	28.39	34.91	74.81
60	1.000	0.631	17.86	4	298.00	21.62	.....	.....
59	0.889	0.933	17.50	3	113.67	31.63	45.23	72.04
58	0.889	0.934	17.52	3	135.67	31.70	50.84	80.72
57	0.889	0.931	17.54	3	146.33	31.60	52.80	84.01
56	0.889	0.925	17.53	3	153.67	31.40	53.32	85.41
55	0.889	0.922	17.44	4	156.50	31.21	52.85	85.62
54	0.889	0.916	17.45	3	160.33	31.00	52.66	85.84
53	0.889	0.909	17.46	3	164.00	30.79	52.35	85.87
52	0.889	0.908	17.47	4	168.00	30.59	52.07	85.92
51	0.889	0.898	17.48	4	172.75	30.42	51.95	86.14
50	0.889	0.893	17.48	4	176.75	30.25	51.52	85.91
49	0.889	0.886	17.49	4	184.00	30.05	51.08	85.69
48	0.889	0.864	17.53	4	206.50	29.31	47.77	81.98
47	0.889	0.795	17.59	4	244.75	27.02	38.97	63.02
46	0.889	0.572	17.91	4	298.75	19.61	.....	.....
45	0.741	0.809	17.57	4	106.00	27.48	39.23	71.55
44	0.741	0.801	17.57	4	139.50	27.20	45.18	83.35
43	0.741	0.791	17.55	3	152.00	26.85	45.71	85.53
41	0.741	0.784	17.57	4	159.75	26.63	45.82	86.35
40	0.741	0.780	17.60	4	166.25	26.52	46.15	87.18
42	0.741	0.777	17.56	4	171.50	26.40	46.02	87.53
39	0.741	0.770	17.64	4	180.75	26.22	45.99	87.68
38	0.741	0.758	17.67	4	191.50	25.82	44.30	85.61
37	0.741	0.727	17.73	3	210.67	24.81	38.99	78.15
36	0.741	0.693	17.79	3	231.33	23.68	32.11	67.30
35	0.741	0.508	18.03	2	298.00	17.31	.....	.....
34	0.593	0.669	17.84	3	101.00	22.80	32.71	70.63
33	0.593	0.660	17.84	3	122.67	22.61	36.32	79.40
31	0.593	0.644	17.86	3	145.00	22.07	37.57	84.04
30	0.593	0.642	17.85	3	151.33	21.98	37.81	84.97
32	0.593	0.637	17.88	4	158.00	21.83	38.01	85.87
29	0.593	0.632	17.91	4	163.75	21.68	37.88	86.02
28	0.593	0.622	17.91	3	173.00	21.33	36.82	84.98
27	0.593	0.606	17.99	3	188.00	20.82	34.79	81.90
26	0.593	0.537	18.16	4	239.00	18.54	22.11	57.92
25	0.593	0.406	18.05	3	281.00	13.98	.....	.....
24	0.444	0.505	17.91	3	94.00	17.32	23.92	67.99
22	0.444	0.489	17.92	2	130.00	16.78	27.06	79.36
21	0.444	0.486	17.94	3	136.00	16.70	27.68	81.48
23	0.444	0.486	17.92	4	139.25	16.66	27.70	81.82
20	0.444	0.481	17.94	3	145.33	16.50	27.57	82.12
19	0.444	0.472	17.94	4	153.00	16.21	26.90	81.56
18	0.444	0.465	17.95	5	161.40	15.98	26.13	80.34
17	0.444	0.450	17.99	4	178.00	15.46	24.71	78.32
16	0.444	0.402	18.04	4	231.50	13.85	16.07	56.70

TABLE 1.—(*Continued.*)

Number of experiment.	Proportional Part of: Percentage of full opening of speed-gate.	Proportional Part of: Percentage of full discharge of wheel.	Head acting on wheel, in feet.	Duration of experiment, in minutes.	Revolutions of wheel per minute.	Quantity of water discharged by wheel, in cubic feet per second.	Horse-power developed by wheel.	Percentage of efficiency of wheel.
15	0.444	0.325	18.17	4	281.25	11.22	.....	.....
14	0.296	0.325	18.19	3	89.67	11.22	15.35	66.32
13	0.296	0.320	18.21	3	107.00	11.06	16.58	72.60
11	0.296	0.317	18.23	3	114.67	10.97	16.98	74.85
12	0.296	0.316	18.22	3	118.33	10.94	16.97	75.07
10	0.296	0.313	18.24	3	122.67	10.82	17.03	76.07
9	0.296	0.308	18.25	4	130.75	10.68	16.94	76.62
8	0.296	0.304	18.26	4	137.75	10.54	16.57	75.91
7	0.296	0.300	18.30	4	146.50	10.41	16.27	75.29
6	0.296	0.295	18.31	4	154.75	10.23	15.75	74.15
5	0.296	0.291	18.32	4	164.00	10.10	15.17	72.31
4	0.296	0.283	18.33	5	181.20	9.82	14.25	69.81
3	0.296	0.276	18.36	4	191.25	9.58	13.27	66.53
2	0.296	0.266	18.33	4	208.00	9.23	11.55	60.18
1	0 296	0.225	18.37	4	261.00	7.83	.....	.....

NOTE.—For Experiments Nos. 1, 15, 25, 35, 46 and 60, the jacket was loose.

During the above experiments, the weight of the dynamometer, and of that portion of the shaft which was above the lowest coupling was 1 300 lb.

With the flume empty, a strain of 1.0 lb., applied at a distance of 2.4 ft. from the center of the shaft, sufficed to start the wheel.

of Pitot tubes for measuring the velocity at the top and bottom of the guides and in the draft-tube. The latter consisted of a movable tube, with velocity and pressure points, introduced through a gland in the side of the draft-tube about 2 ft. below the bottom of the runner. This tube was arranged to move horizontally across the diameter of the draft-tube and also to revolve about its own axis. By means of these two movements, the velocity at any distance from the center of the shaft could be determined, and also the angle of whirl. The latter would necessarily be the angle of the velocity point which gave the highest reading. Both movements of the tube were controlled from the floor above, and, by means of indicators, the exact position of the point was at all times known.

Fig. 2, Plate XXXI, is a photograph of the operating floor, showing the brake for measuring the power of the wheel. At the left is the gauge-board and auxiliary apparatus connected to the Pitot tubes below. The water columns were drawn up into view by means of a vacuum. The arrangement of this apparatus is shown diagrammatically in Fig. 3. The position of the columns in the glass tubes was controlled

TABLE 2.—TESTS OF A 28-INCH R. H. WELLMAN-SEAVER-MORGAN COMPANY TURBINE WHEEL, No. 1796.

Date, February 25th, 1909.

Wheel supported by ball-bearing step. Swing-gate. Conical draft-tube.

Number of experiment.	Proportional Part of:		Head acting on wheel, in feet.	Duration of experiment, in minutes.	Revolutions of wheel per minute.	Quantity of water discharged by wheel, in cubic feet per second.	Horse-power developed by wheel.	Percentage of efficiency of wheel.
	Percentage of full opening of speed-gate.	Percentage of full discharge of wheel.						
95	1.077	0.971	17.11	4	153.00	97.00	125.20	66.52
94	1.077	1.017	16.97	3	199.67	101.16	147.66	75.84
93	1.077	1.036	16.94	3	224.33	102.98	156.38	79.04
92	1.077	1.053	16.89	3	239.33	104.50	159.58	79.72
91	1.077	1.061	16.87	3	247.33	105.22	161.17	80.06
89	1.077	1.068	16.81	3	253.67	105.70	161.45	80.12
90	1.077	1.072	16.80	3	259.00	106.08	161.71	80.01
88	1.077	1.079	16.82	4	267.50	106.82	162.15	79.58
87	1.077	1.026	17.05	3	294.67	102.27	125.03	63.23
86	1.000	0.918	17.28	3	147.00	91.65	120.29	66.98
85	1.000	0.957	17.16	2	190.50	95.70	144.34	77.50
84	1.000	0.972	17.14	3	211.67	97.10	152.69	80.89
83	1.000	0.981	17.13	3	225.00	98.03	156.85	82.36
82	1.000	0.990	17.11	3	232.67	98.90	157.96	82.31
80	1.000	0.996	17.09	4	240.25	99.43	160.19	83.13
81	1.000	1.003	17.07	3	247.33	100.07	161.17	83.19
79	1.000	1.004	17.07	4	252.25	100.14	160.55	82.82
78	1.000	1.001	17.13	4	259.00	100.00	157.00	80.82
77	1.000	0.988	17.22	3	268.33	98.43	146.39	76.15
76	1.000	0.911	17.47	4	293.50	91.96	106.75	58.59
106	0.923	0.868	17.32	4	143.25	87.24	115.49	67.39
105	0.923	0.899	17.24	5	176.00	90.12	135.49	76.90
104	0.923	0.920	17.15	4	201.00	91.96	146.21	81.74
101	0.923	0.931	16.93	5	213.20	92.52	148.62	83.66
100	0.923	0.936	16.93	6	220.67	92.96	150.48	84.31
99	0.923	0.942	16.91	4	227.25	93.51	151.52	84.50
102	0.923	0.945	16.93	4	232.00	93.82	151.88	84.31
103	0.923	0.945	17.04	4	235.50	94.14	152.74	83.96
98	0.923	0.945	16.93	4	237.75	93.82	151.32	84.00
97	0.923	0.924	17.02	4	254.50	92.04	138.84	78.15
96	0.923	0.823	17.25	4	288.25	82.47	87.36	54.15
42	0.923	0.870	17.17	3	146.33	87.02	116.20	68.57
41	0.923	0.895	17.10	4	170.00	89.37	130.87	75.51
40	0.923	0.921	17.04	4	202.75	91.80	146.25	82.44
39	0.923	0.928	16.99	3	208.67	92.35	147.99	83.17
35	0.923	0.932	17.03	4	216.25	92.81	150.75	84.10
38	0.923	0.937	16.97	4	220.00	93.20	151.36	84.38
36	0.923	0.939	17.01	4	223.75	93.44	152.58	84.65
34	0.923	0.940	17.02	4	226.25	93.66	150.86	83.45
37	0.923	0.944	16.97	3	238.33	93.90	151.69	83.94
33	0.923	0.921	17.13	4	256.25	92.05	139.80	78.18
32	0.923	0.823	17.27	3	288.00	82.54	87.29	53.99
31	0.923	0.730	17.50	4	334.75	73.75	.....	.....
74	0.846	0.824	17.46	3	158.67	83.15	120.23	73.02
75	0.846	0.836	17.46	3	175.67	84.35	129.91	77.78
72	0.846	0.861	17.34	5	202.20	86.50	143.40	84.30
70	0.846	0.865	17.33	4	209.00	86.95	145.69	85.25
71	0.846	0.868	17.33	3	215.00	87.24	147.27	85.89
73	0.846	0.870	17.34	4	219.25	87.47	148.19	86.15
69	0.846	0.869	17.32	4	221.25	87.32	147.53	86.01
68	0.846	0.866	17.33	4	227.75	87.02	144.96	84.76
67	0.846	0.858	17.36	4	231.75	86.25	140.48	82.73
66	0.846	0.845	17.39	4	243.75	85.11	132.98	79.22
65	0.846	0.828	17.44	4	256.50	83.44	124.39	75.37

TABLE 2.—(*Continued.*)

Number of experiment.	Proportional Part of: Percentage of full opening of speed-gate.	Proportional Part of: Percentage of full discharge of wheel.	Head acting on wheel, in feet.	Duration of experiment, in minutes.	Revolutions of wheel per minute.	Quantity of water discharged by wheel, in cubic feet per second.	Horse-power developed by wheel.	Percentage of efficiency of wheel.
64	0.846	0.754	17.59	3	282.00	76.31	85.47	56.15
30	0.769	0.750	17.38	3	141.00	75.52	102.56	68.90
26	0.769	0.766	17.35	3	166.00	77.02	115.72	76.86
27	0.769	0.779	17.32	3	183.00	78.24	124.24	80.84
25	0.769	0.789	17.31	3	194.00	79.25	129.36	83.15
29	0.769	0.793	17.25	4	200.75	79.48	131.42	84.52
28	0.769	0.792	17.26	4	206.00	79.40	131.11	84.36
24	0.769	0.773	17.38	4	226.25	77.82	123.43	80.47
23	0.769	0.735	17.49	3	251.33	74.17	106.64	72.49
22	0.769	0.690	17.56	3	269.67	69.82	81.73	58.78
21	0.769	0.623	17.68	4	328.75	63.27	.....	.....
20	0.615	0.617	17.91	4	139.50	63.07	88.79	69.31
16	0.615	0.627	17.79	3	158.33	63.81	95.97	74.55
17	0.615	0.634	17.80	3	171.33	64.55	101.26	77.71
15	0.615	0.638	17.73	4	179.50	64.82	103.37	79.31
18	0.615	0.636	17.77	4	183.00	64.74	103.16	79.07
19	0.615	0.634	17.80	2	188.00	64.61	102.56	78.64
14	0.615	0.627	17.72	4	194.50	63.74	100.21	78.24
13	0.615	0.596	17.77	3	218.67	60.60	92.78	75.97
12	0.615	0.563	17.79	4	243.00	57.29	73.65	63.72
11	0.615	0.519	17.92	4	312.00	53.00	.....	.....
10	0.462	0.452	17.34	3	117.33	45.40	56.90	63.73
5	0.462	0.453	17.02	3	136.00	45.65	61.83	70.17
7	0.462	0.461	17.08	4	146.75	46.04	64.94	72.82
6	0.462	0.462	17.05	4	152.00	46.04	66.34	74.52
4	0.462	0.462	16.92	4	155.25	45.90	65.88	74.79
9	0.462	0.459	17.16	3	162.00	45.94	66.78	74.69
8	0.462	0.457	17.15	3	166.67	45.69	65.67	73.90
3	0.462	0.451	16.98	4	172.25	44.83	62.65	72.57
2	0.462	0.432	17.05	4	217.50	43.04	52.74	63.37
1	0.462	0.404	17.18	5	282.40	40.40	.....	.....

NOTE.—During the above experiments, the weight of the dynamometer, and of that portion of the shaft which was above the lowest coupling, was 2 600 lb.

With the flume empty, a strain of 0.5 lb., applied at a distance of 3.2 ft. from the center of the shaft, sufficed to start the wheel.

by three-way cocks. By connecting a set of tubes with the vacuum tank, the columns were drawn above the floor level, after which, by closing the cock, they were held in a stationary position. If drawn too high they could be dropped back by opening the cock to the atmosphere. After the columns were once adjusted and the cocks closed, they would stand indefinitely without attention.

This apparatus for the measurement of velocities has been described at some length for the benefit of those who may be interested in the same line of experiments. Lack of space precludes the publication of any of the data obtained, although they are of great interest to the designer, and throw much light on some very obscure points in the theory of turbine design.

TABLE 3.—TESTS OF A 30-INCH R. H. WELLMAN-SEAVER-MORGAN COMPANY TURBINE WHEEL, No. 1797.

Date, February 26th and 27th, 1909.

Wheel supported by ball-bearing step. Swing-gate. Conical draft-tube.

Number of experiment.	Proportional Part of: Percentage of full opening of speed-gate.	Proportional Part of: Percentage of full discharge of wheel.	Head acting on wheel, in feet.	Duration of experiment, in minutes.	Revolutions of wheel per minute.	Quantity of water discharged by wheel, in cubic feet per second.	Horse-power developed by wheel.	Percentage of efficiency of wheel.
82	1.000	0.942	17.06	3	149.00	94.89	126.41	68.85
81	1.000	0.950	16.99	3	160.33	95.52	131.16	71.26
80	1.000	0.963	17.00	4	190.50	96.85	144.30	77.28
75	1.000	0.977	17.02	4	210.50	98.34	150.52	79.30
74	1.000	0.984	16.95	4	220.00	98.81	153.31	80.72
76	1.000	0.992	17.01	4	227.25	99.75	155.61	80.87
78	1.000	0.994	16.94	4	229.50	99.75	155.76	81.28
77	1.000	0.997	16.98	4	231.25	100.14	155.55	80.66
79	1.000	1.002	16.90	3	239.67	100.45	156.85	81.47
73	1.000	1.003	16.94	4	254.75	100.70	154.37	79.80
72	1.000	0.871	17.13	3	287.00	87.93	86.96	50.91
71	1.000	0.728	17.44	3	317.67	74.17	.....	.....
52	0.923	0.875	17.08	3	131.33	88.14	111.42	65.26
51	0.923	0.896	16.95	3	156.33	89.97	127.89	73.95
50	0.923	0.913	16.93	4	188.25	91.66	142.59	81.02
44	0.923	0.920	16.89	4	201.25	92.20	146.34	82.86
45	0.923	0.927	16.85	4	211.50	92.81	148.67	83.83
46	0.923	0.931	16.86	4	217.00	93.20	149.91	84.12
49	0.923	0.934	16.88	4	220.50	93.42	150.99	84.68
47	0.923	0.936	16.83	4	223.75	93.60	151.86	85.00
48	0.923	0.937	16.83	3	226.33	93.74	150.87	84.32
43	0.923	0.929	16.88	4	241.75	93.13	146.49	82.17
42	0.923	0.788	17.21	4	276.25	79.76	83.70	53.77
41	0.923	0.670	17.44	4	312.50	68.26	.....	.....
70	0.846	0.868	17.07	4	192.75	87.47	142.50	84.15
67	0.846	0.875	17.10	4	205.00	88.22	147.83	86.41
68	0.846	0.876	17.06	4	209.50	88.22	148.53	87.02
66	0.846	0.876	17.11	4	212.00	88.37	149.02	86.91
69	0.846	0.877	17.05	3	213.00	88.30	148.43	86.94
65	0.846	0.876	17.13	5	213.80	88.37	147.70	86.03
64	0.846	0.875	17.13	5	215.80	88.30	146.46	85.38
63	0.808	0.810	17.33	4	143.50	82.24	114.78	71.01
62	0.808	0.834	17.25	3	177.00	84.44	135.14	81.81
61	0.808	0.843	17.24	5	194.80	85.35	142.83	85.59
60	0.808	0.847	17.21	3	201.00	85.73	144.94	86.62
59	0.808	0.848	17.19	3	206.00	85.73	146.05	87.39
58	0.808	0.847	17.19	4	208.25	85.65	145.12	86.91
57	0.808	0.845	17.19	3	210.00	85.41	143.80	86.36
56	0.808	0.840	17.22	3	216.67	85.05	141.80	85.37
55	0.808	0.825	17.25	4	224.75	83.53	136.19	83.34
54	0.808	0.807	17.29	4	236.50	81.80	128.98	80.41
53	0.808	0.695	17.51	4	264.25	70.97	80.06	56.81
40	0.769	0.755	17.29	4	118.50	76.53	93.35	62.21
39	0.769	0.776	17.21	4	146.50	78.47	110.97	72.46
35	0.769	0.800	17.08	4	174.75	80.49	127.07	81.74
36	0.769	0.806	17.03	4	190.75	81.13	135.24	86.31
37	0.769	0.805	17.10	4	197.25	81.21	137.46	87.28
38	0.769	0.805	17.15	4	200.50	81.28	137.29	86.84
34	0.769	0.803	16.83	4	199.50	80.34	132.98	86.72
33	0.769	0.788	16.93	4	211.75	79.04	128.32	84.55
32	0.769	0.745	16.93	4	232.50	74.72	112.71	78.56
31	0.769	0.660	17.36	4	257.50	67.02	78.02	59.18
30	0.769	0.579	17.60	3	306.00	59.27	.....	.....
29	0.615	0.610	17.69	3	100.67	62.60	69.54	55.37

TABLE 3.—(*Continued.*)

Number of experiment.	Proportional Part of: Percentage of full opening of speed-gate.	Proportional Part of: Percentage of full discharge of wheel.	Head acting on wheel, in feet.	Duration of experiment, in minutes.	Revolutions of wheel, per minute.	Quantity of water discharged by wheel, in cubic feet per second.	Horse-power developed by wheel.	Percentage of efficiency of wheel.
28	0.615	0.621	17.64	3	127.67	63.60	85.10	66.89
35	0.615	0.643	17.60	3	167.33	65.76	104.44	79.57
27	0.615	0.644	17.57	4	175.50	65.83	107.41	81.89
24	0.615	0.643	17.61	4	178.00	65.76	107.86	82.13
26	0.615	0.641	17.59	3	179.67	65.55	106.70	81.60
23	0.615	0.482	17.96	3	296.67	49.84	.....	.....
20	0.615	0.634	17.88	4	155.00	65.37	101.44	76.53
21	0.615	0.634	17.80	5	151.40	65.22	98.17	74.56
22	0.615	0.639	17.69	3	160.33	65.50	102.98	78.37
19	0.615	0.644	18.08	5	174.20	66.80	111.89	81.69
18	0.615	0.643	18.11	3	180.00	66.73	112.35	81.97
17	0.615	0.638	18.07	4	183.50	66.19	111.20	81.98
16	0.615	0.636	18.03	4	185.00	65.90	109.86	81.53
15	0.615	0.630	17.96	3	190.00	65.15	109.38	82.43
14	0.615	0.609	17.81	4	201.00	62.66	103.53	81.80
13	0.615	0.574	17.79	4	216.75	59.07	91.94	77.15
12	0.615	0.553	17.54	5	245.20	56.46	74.29	66.15
11	0.615	0.483	17.71	5	294.60	49.58	.....	.....
8	0.462	0.475	17.82	3	104.33	48.95	58.16	58.80
7	0.462	0.483	17.75	4	139.75	49.58	71.98	72.12
6	0.462	0.482	17.76	3	147.00	49.51	73.93	74.14
5	0.462	0.481	17.80	4	156.75	49.45	75.99	76.12
9	0.462	0.477	17.93	2	162.00	49.22	76.57	76.51
10	0.462	0.472	17.97	3	167.00	48.78	75.90	76.35
4	0.462	0.464	17.88	5	173.60	47.80	73.64	75.97
3	0.462	0.436	18.05	4	216.50	45.13	65.60	71.01
2	0.462	0.415	18.08	4	246.25	43.02	44.77	50.75
1	0.462	0.381	18.27	4	280.50	39.75	.....	.....

NOTE.—The jacket was loose for Experiments Nos. 1, 11, 23, 30, 41, and 71.

During the above experiments, the weight of the dynamometer, and of that portion of the shaft which was above the lowest coupling was 2 600 lb.

With the flume empty, a strain of 0.5 lb., applied at a distance of 3.2 ft. from the center of the shaft, sufficed to start the wheel.

Figs. 4 to 19, inclusive, are performance curves plotted from the test reports, on the common basis of 1 ft. head. The head at the testing flume fluctuates, and it is therefore necessary to reduce all readings to a common basis. With constant efficiency, the brake-horse-power varies as $\sqrt{h^3}$, and the speed and discharge as $\sqrt{h}$. This principle is true, theoretically, and has been repeatedly proven, experimentally, and all the computations have been made upon this basis.

Three sets of curves have been plotted for each of the five wheels. The first in order for each wheel is a set of brake-horse-power and efficiency curves for all gate openings plotted to speed. Each curve is designated by the gate opening which it represents. In addition to power and speed, the writer usually plots the discharge curves, but

TABLE 4.—TESTS OF A 31-INCH R. H. WELLMAN-SEAVER-MORGAN COMPANY TURBINE WHEEL, No. 1799.

Date, March 2d and 3d, 1909.

Wheel supported by ball-bearing step. Swing-gate. Conical draft-tube.

Number of experiment.	Proportional Part of: Percentage of full opening of speed-gate.	Proportional Part of: Percentage of full discharge of wheel.	Head acting on wheel, in feet.	Duration of experiment, in minutes.	Revolutions of wheel per minute.	Quantity of water discharged by wheel, in cubic feet per second.	Horse power developed by wheel.	Percentage of efficiency of wheel.
49	1.000	1.049	17.15	3	134.67	79.75	115.81	74.66
48	1.000	1.038	17.15	3	147.00	78.95	119.29	77.68
47	1.000	1.025	17.19	3	165.67	78.02	123.40	81.13
46	1.000	1.024	17.18	2	174.00	77.91	124.34	81.91
45	1.000	1.018	17.19	3	178.33	77.52	124.19	82.18
44	1.000	1.013	17.19	3	188.33	77.16	124.35	82.66
43	1.000	0.012	17.19	4	186.25	77.09	124.07	82.56
42	1.000	1.009	17.18	4	189.50	76.82	123.94	82.81
41	1.000	1.007	17.19	3	193.00	76.67	123.89	82.89
40	1.000	1.006	17.12	4	195.50	76.44	123.13	82.96
39	1.000	1.002	17.09	4	200.25	76.10	122.48	83.04
38	1.000	0.999	17.10	4	206.25	75.87	122.41	83.19
37	1.000	0.997	17.07	3	210.33	75.66	121.01	82.62
36	1.000	0.993	17.09	4	219.00	75.38	119.36	81.70
35	1.000	0.906	17.24	4	258.75	69.11	78.35	57.98
34	1.000	0.744	17.45	4	302.25	57.07	.....	.....
32	0.883	0.909	17.18	3	129.33	69.23	101.03	74.90
31	0.883	0.904	17.20	4	154.25	68.82	111.16	82.81
33	0.883	0.901	17.19	4	165.00	68.60	113.91	85.18
30	0.883	0.896	17.20	3	172.33	68.25	114.80	86.28
29	0.883	0.892	17.21	4	181.50	67.97	116.51	87.82
27	0.883	0.888	17.25	3	188.00	67.77	117.27	88.45
26	0.883	0.886	17.30	4	194.00	67.71	117.49	88.44
28	0.883	0.885	17.23	4	197.25	67.43	117.06	88.85
25	0.883	0.883	17.31	4	201.75	67.49	116.07	87.61
24	0.883	0.857	17.28	4	213.25	65.42	109.77	85.62
23	0.883	0.818	17.31	4	227.00	62.53	96.23	78.39
22	0.883	0.761	17.42	4	244.25	58.32	73.96	64.19
21	0.883	0.620	17.65	4	291.75	47.83	.....	.....
69	0.750	0.835	17.28	4	124.75	63.73	91.41	73.19
68	0.750	0.839	17.23	4	148.75	63.93	103.59	82.93
67	0.750	0.834	17.23	4	168.75	63.60	109.35	87.99
64	0.750	0.830	17.24	4	179.25	63.26	110.72	89.52
65	0.750	0.829	17.23	4	182.00	63.20	110.77	89.69
63	0.750	0.828	17.25	4	186.25	63.12	111.66	90.43
66	0.750	0.824	17.23	3	188.67	62.85	110.83	90.24
62	0.750	0.821	17.29	4	191.75	62.72	110.32	89.70
61	0.750	0.809	17.30	4	197.50	61.81	107.64	88.76
60	0.750	0.796	17.34	4	203.25	60.86	104.62	87.42
59	0.750	0.768	17.38	4	212.50	58.80	96.52	83.28
58	0.750	0.693	17.55	4	237.00	53.31	71.76	67.64
57	0.750	0.573	17.75	4	287.00	44.32	.....	.....
20	0.667	0.726	17.27	4	106.25	55.43	70.78	65.20
19	0.667	0.737	17.25	3	148.67	56.25	90.93	82.59
18	0.667	0.739	17.25	4	161.00	56.39	95.55	86.62
17	0.667	0.738	17.24	4	168.50	56.31	96.94	88.05
16	0.667	0.733	17.27	3	173.67	55.92	96.76	88.35
15	0.667	0.722	17.30	4	179.25	55.17	95.52	88.25
14	0.667	0.690	17.34	4	189.50	53.48	91.81	87.30
13	0.667	0.671	17.39	4	201.75	51.37	85.52	84.42
11	0.667	0.508	17.63	4	280.75	39.17	.....	.....
8	0.500	0.554	17.82	3	117.00	42.95	60.23	69.39
7	0.500	0.546	17.91	3	135.00	42.46	65.40	75.84
10	0.500	0.548	17.67	3	151.00	42.28	68.58	80.95

TABLE 4.—(*Continued.*)

Number of experiment.	Proportional Part of: Percentage of full opening of speed-gate.	Proportional Part of: Percentage of full discharge of wheel.	Head acting on wheel, in feet.	Duration of experiments, in minutes.	Revolutions of wheel per minute.	Quantity of water discharged by wheel, in cubic feet per second.	Horse-power developed by wheel.	Percentage of efficiency of wheel.
6	0.500	0.548	18.05	4	157.50	42.72	71.54	81.80
9	0.500	0.547	17.71	4	157.00	42.28	69.41	81.73
5	0.500	0.539	18.12	5	167.60	42.11	71.05	82.10
4	0.500	0.512	18.13	4	187.00	40.02	67.95	82.58
3	0.500	0.488	18.18	4	213.00	38.21	68.05	73.68
2	0.500	0.460	18.07	4	232.00	35.92	42.15	57.26
1	0.500	0.402	18.20	3	275.00	31.50	.....	.....
52	0.333	0.362	18.18	3	96.00	28.35	34.88	59.68
55	0.333	0.361	18.01	3	117.33	28.10	39.08	68.09
51	0.333	0.348	18.19	4	133.00	27.25	40.27	71.64
54	0.333	0.347	18.21	3	139.67	27.20	40.60	72.28
53	0.333	0.340	18.22	3	143.67	26.62	39.15	71.18
56	0.333	0.333	18.06	4	148.00	26.00	37.64	70.69
50	0.333	0.316	18.26	3	201.67	24.82	36.64	71.98

NOTE.—For Experiments Nos. 1, 11, 21, 34, and 57, the jacket was loose.

During the above experiments, the weight of the dynamometer and of that portion of the shaft which was above the lowest coupling was 2 600 lb.

With the flume empty, a strain of 1.0 lb., applied at a distance of 3.2 ft. from the center of the shaft, sufficed to start the wheel.

they have been omitted from this paper in order to economize space.* When included, such a diagram might be termed in common parlance a "personality chart," because it embodies all the characteristics and peculiarities of the wheel. From it separate performance curves can be plotted for any speed whatsoever within the range of the test. The heavy speed line drawn through each of these charts indicates the "normal speed" of the wheel, that is, the speed at which it gives its maximum efficiency.

The second set of curves shows the performance of the wheels along these heavy lines, or at normal speed. The gate openings are also plotted to discharge, so that it is possible to read the efficiency at any fractional part of the full gate opening as well as at any fractional part of the full discharge of the wheel.

The first set of curves shows, in addition to the information already referred to, a scale of values of ϕ, in the equation,

$$W_1 = \phi \sqrt{2\ gh},$$

where W_1 is the peripheral speed of the wheel, based on its rated

* For full details of this method of plotting test curves, see Chapter XVI of "Water Power Engineering," by Daniel W. Mead, M. Am. Soc. C. E.

TABLE 5.—Tests of a 31-inch R. H. Wellman-Seaver-Morgan Company Turbine Wheel, No. 1800.

Date, March 4th and 5th, 1909.

Wheel supported on ball-bearing step. Swing-gate. Conical draft-tube.

Number of experiment.	Proportional Part of:		Head acting on wheel, in feet.	Duration of experiment, in minutes.	Revolutions of wheel per minute.	Quantity of water discharged by wheel, in cubic feet per second.	Horse-power developed by wheel.	Percentage of efficiency of wheel.
	Percentage of full opening of speed-gate.	Percentage of full discharge of wheel.						
65	1.000	1.052	17.38	3	112.33	64.87	82.99	64.91
64	1.000	1.040	17.41	3	133.33	64.20	91.24	71.98
63	1.000	1.025	17.43	4	154.25	63.33	96.21	76.85
62	1.000	1.014	17.42	3	169.67	62.60	98.64	79.76
61	1.000	1.009	17.40	3	183.33	62.25	99.92	81.34
60	1.000	1.006	17.42	4	193.25	62.13	100.65	82.00
59	1.000	1.004	17.43	4	201.00	62.01	101.03	82.42
58	1.000	1.002	17.44	4	207.00	61.88	101.54	82.96
56	1.000	1.000	17.35	4	211.00	61.65	100.95	83.22
57	1.000	0.999	17.39	4	216.75	61.65	101.07	83.13
55	1.000	0.998	17.36	4	221.50	61.53	100.60	83.05
54	1.000	0.995	17.38	4	247.50	61.40	97.42	80.50
53	1.000	0.951	17.45	4	266.75	58.80	80.77	69.41
52	1.000	0.730	17.71	4	321.25	45.44	.....	.....
51	0.883	0.889	17.48	4	132.25	55.02	79.29	72.69
50	0.883	0.888	17.48	4	147.00	54.91	83.68	76.88
49	0.883	0.885	17.47	4	165.00	54.71	88.93	82.04
48	0.883	0.879	17.39	4	179.25	54.24	91.18	85.24
47	0.883	0.876	17.42	5	189.20	54.11	91.66	85.75
44	0.883	0.873	17.54	4	197.50	54.11	93.29	86.67
43	0.883	0.873	17.40	4	204.25	53.85	92.77	87.30
45	0.883	0.872	17.40	3	209.00	53.79	92.40	87.05
46	0.883	0.871	17.42	4	215.50	53.79	92.66	87.19
42	0.883	0.858	17.40	3	225.00	52.92	88.57	84.81
41	0.883	0.810	17.45	3	248.00	50.06	75.09	75.80
40	0.883	0.609	17.68	3	314.00	37.87	.....	.....
75	0.733	0.801	17.76	4	158.50	49.93	81.59	81.13
73	0.733	0.798	17.73	3	183.33	49.70	86.60	86.66
72	0.733	0.797	17.71	3	190.33	49.63	87.60	87.88
71	0.733	0.796	17.72	3	197.00	49.57	88.28	88.62
74	0.733	0.795	17.73	3	199.00	49.50	87.97	88.39
70	0.733	0.793	17.72	3	201.67	49.37	87.93	88.63
69	0.733	0.788	17.70	4	204.50	49.02	86.69	88.10
68	0.733	0.775	17.71	4	213.50	48.27	84.04	86.69
67	0.733	0.722	17.79	3	234.00	45.04	70.85	77.97
66	0.733	0.544	18.03	3	308.67	34.15	.....	.....
38	0.667	0.751	17.58	3	171.00	46.58	78.70	84.75
37	0.667	0.750	17.57	4	184.00	46.52	81.34	87.75
39	0.667	0.749	17.56	4	187.00	46.45	81.54	88.14
35	0.667	0.749	17.59	4	185.50	46.45	80.88	87.29
36	0.667	0.747	17.59	4	188.75	46.38	81.73	88.33
34	0.667	0.742	17.58	4	192.50	45.02	80.44	87.67
33	0.667	0.731	17.59	4	198.25	45.37	78.04	86.22
32	0.667	0.753	17.70	3	127.67	46.87	68.04	72.32
31	0.667	0.752	17.68	3	144.67	46.76	72.72	77.56
29	0.667	0.752	17.66	4	164.25	46.76	77.59	82.85
30	0.667	0.753	17.68	3	178.67	46.82	81.15	86.44
28	0.667	0.750	17.62	3	189.00	46.57	82.41	88.56
27	0.667	0.741	17.62	3	193.67	46.02	80.93	88.00
26	0.667	0.731	17.64	3	201.00	45.44	79.12	87.04
25	0.667	0.725	17.63	3	204.00	45.05	77.88	86.41
24	0.667	0.709	17.64	3	213.33	44.05	74.93	85.03
23	0.667	0.678	17.70	4	224.50	42.23	67.98	80.19
22	0.667	0.506	17.85	3	303.33	31.61	.....	.....

TABLE 5.—(*Continued.*)

Number of experiment.	Proportional Part of: Percentage of full opening of speed-gate.	Proportional Part of: Percentage of full discharge of wheel.	Head acting on wheel, in feet.	Duration of experiment, in minutes.	Revolutions of wheel per minute.	Quantity of water discharged by wheel, in cubic feet per second.	Horse-power developed by wheel.	Percentage of efficiency of wheel.
21	0.500	0.566	17.72	2	124.00	35.27	50.31	70.98
20	0.500	0.560	17.74	3	136.33	34.87	52.84	75.32
19	0.500	0.554	17.71	4	146.50	34.50	53.23	76.82
18	0.500	0.543	17.72	4	155.75	33.82	52.82	77.72
17	0.500	0.535	17.74	3	166.00	33.31	52.27	78.00
16	0.500	0.521	17.77	3	176.67	32.47	51.36	78.48
15	0.500	0.507	17.79	3	189.50	31.61	50.36	78.97
14	0.500	0.494	17.83	4	200.25	30.87	48.51	77.71
13	0.500	0.481	17.86	3	214.00	30.08	45.36	74.45
12	0.500	0.464	17.89	4	228.75	29.02	41.56	70.58
11	0.500	0.380	18.00	4	299.50	23.87	.....	.....
8	0.333	0.360	18.09	4	112.00	22.65	30.52	65.68
7	0.333	0.351	18.12	3	127.00	22.10	31.53	69.43
6	0.333	0.350	18.13	3	136.00	22.02	32.12	70.95
5	0.333	0.346	18.22	4	141.50	21.84	31.71	70.26
4	0.333	0.340	18.24	4	148.00	21.47	31.37	70.63
3	0.333	0.332	18.31	4	154.75	21.03	30.93	70.82
2	0.333	0.323	18.25	4	165.25	20.42	30.02	71.04
10	0.333	0.307	18.17	4	212.50	19.33	28.31	71.08
9	0.333	0.300	18.15	3	231.33	18.93	25.22	64.72
1	0.333	0.248	18.35	4	287.25	15.74	.....	.....

NOTE.—The jacket was loose for Experiments Nos. 1, 11, 22, 40, 52, and 66.

During the above experiments, the weight of the dynamometer and of that portion of the shaft which was above the lowest coupling was 2 600 lb.

With the flume empty, a strain or 0.5 lb., applied at a distance of 3.2 ft. from the center of the shaft, sufficed to start the wheel.

diameter. The third set of curves shows the performance of the wheels at speeds corresponding to several different values of ϕ. When a wheel operates at constant speed under a fluctuating head, the value of ϕ is continually changing. A comparison of these curves, therefore, is a very instructive study, because it shows how the efficiency varies as the head changes. The change in efficiency corresponding to a given change in ϕ is very different for different types of runner.

High-Speed Wheels.—In the use of the term "high-speed wheel," there seems to be much confusion. In the sense in which it is used here, the term does not apply necessarily to wheels of high peripheral speed, although a high value of ϕ is usually an attribute of a "high-speed wheel." As the term is used here, the wheel which, power for power, under a constant head, runs at the highest number of revolutions, is the highest-speed wheel. A high-speed wheel is one which will develop any given quantity of power at a high speed, relative to other wheels developing the same power under the same head. It is not

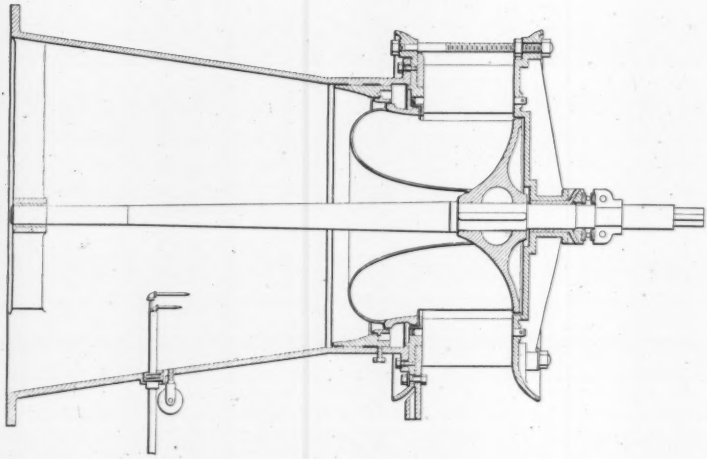


Fig. 1.

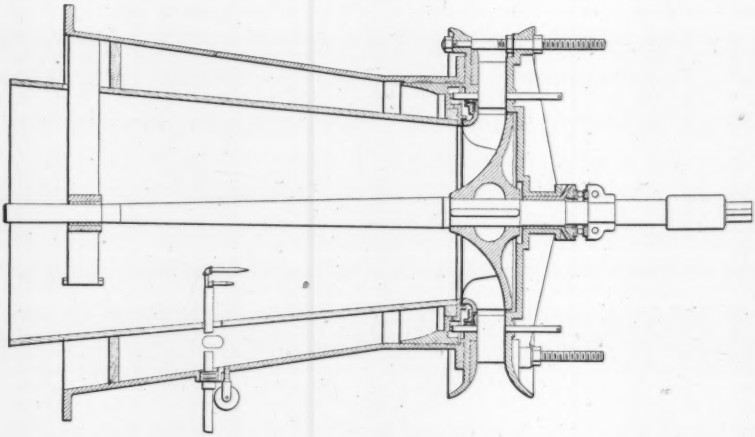


Fig. 2.

possible to compare the speeds of wheels upon the basis of diameter, because the power is not a consistent function of the diameter, except in wheels of homologous design. To determine which of two wheels is the higher-speed, it is usually necessary, unless they differ widely, to reduce them to a common basis of power and head.

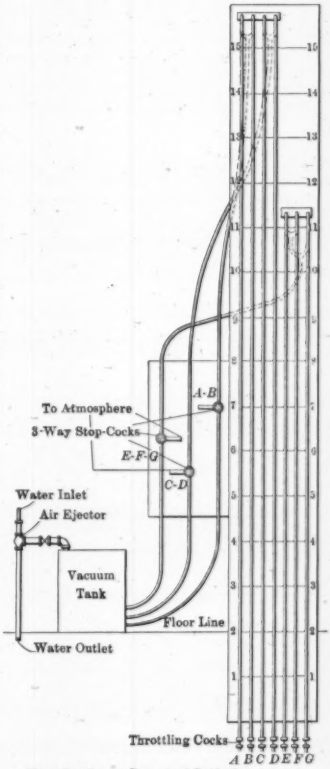
Suppose, for example, that two types of runner, A and B, are to be compared, and it is known from tests, or otherwise, that some certain size of A (the diameter of which need not be given) will develop a maximum of 2 080 h.p. at 500 rev. per min. under 100 ft. head, whereas an entirely different size of B will develop a maximum of 4 590 h.p. at 580 rev. per min. under 150 ft. head. It is impossible to determine from an inspection of these data which of these types is the higher-speed.

They should first be reduced to a common head, say 1 ft. Dividing the horse-power in each case by $\sqrt{h^3}$ and the speed by $\sqrt{h}$, the result is

A. .2.07 h.p. at 50 rev. per min.
B. .2.50 h.p. at 47.4 rev. per min.

A runs at a higher speed than B, but B develops more power than A. One cannot be sure but that if B were reduced in diameter until it developed the same power as A, it might run at as high a speed as A, or higher.

DIAGRAM OF GAUGE-BOARD AND AUXILIARY VACUUM APPARATUS FOR MEASURING VELOCITIES THROUGH GUIDE OPENINGS AND DRAFT-TUBE



A–Pitot Reading at Bottom of Guide Opening
B–Pressure " " " " " "
C–Pitot " " Top " " "
D–Pressure " " " " " "
E–Pitot " from Movable Point in Draft-Tube
F–Pressure " " " " " "
G–Pressure " " Wall of Draft-Tube at same Elevation as Movable Point.

FIG. 3.

Next reduce them to the same power, say 1 h.p. It is known that the power of wheels of homologous design varies directly as the

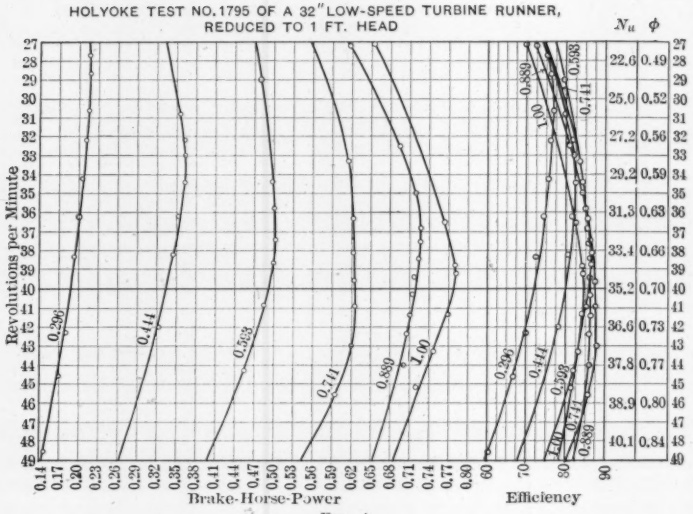

HOLYOKE TEST NO. 1795 OF A 32" LOW-SPEED TURBINE RUNNER,
REDUCED TO 1 FT. HEAD
N_u ϕ
22.6 0.49
25.0 0.52
27.2 0.56
29.2 0.59
31.3 0.63
33.4 0.66
35.2 0.70
36.6 0.73
37.8 0.77
38.9 0.80
40.1 0.84
Revolutions per Minute
Brake-Horse-Power
Efficiency
0.296
0.444
0.593
0.741
0.889
1.00

FIG. 4.

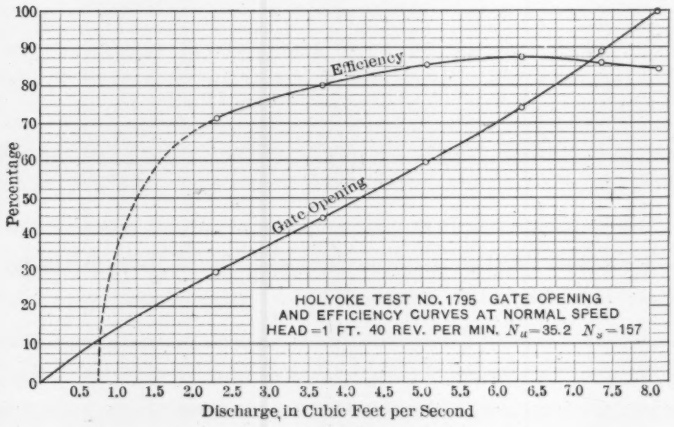

Efficiency
Gate Opening
HOLYOKE TEST NO. 1795 GATE OPENING
AND EFFICIENCY CURVES AT NORMAL SPEED
HEAD = 1 FT. 40 REV. PER MIN. N_u = 35.2 N_s = 157
Percentage
Discharge in Cubic Feet per Second

FIG. 5.

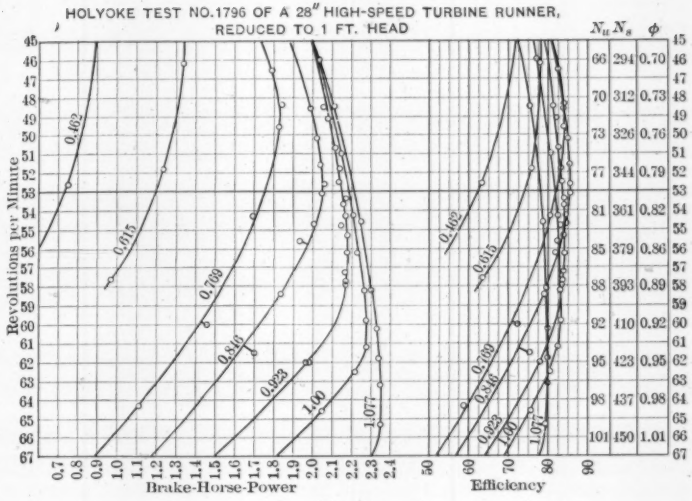

HOLYOKE TEST NO. 1796 OF A 28" HIGH-SPEED TURBINE RUNNER,
REDUCED TO 1 FT. HEAD
N_u N_s ϕ
66 294 0.70
70 312 0.73
73 326 0.76
77 344 0.79
81 361 0.82
85 379 0.86
88 393 0.89
92 410 0.92
95 423 0.95
98 437 0.98
101 450 1.01
0.462
0.615
0.709
0.816
0.923
1.00
1.077
Revolutions per Minute
Brake-Horse-Power
Efficiency

FIG. 6.

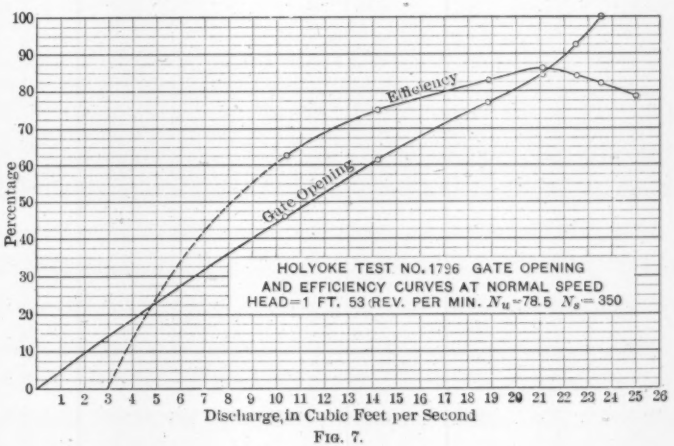

Efficiency
Gate Opening
HOLYOKE TEST NO. 1796 GATE OPENING
AND EFFICIENCY CURVES AT NORMAL SPEED
HEAD=1 FT. 53 REV. PER MIN. N_u=78.5 N_s=350
Percentage
Discharge, in Cubic Feet per Second

FIG. 7.

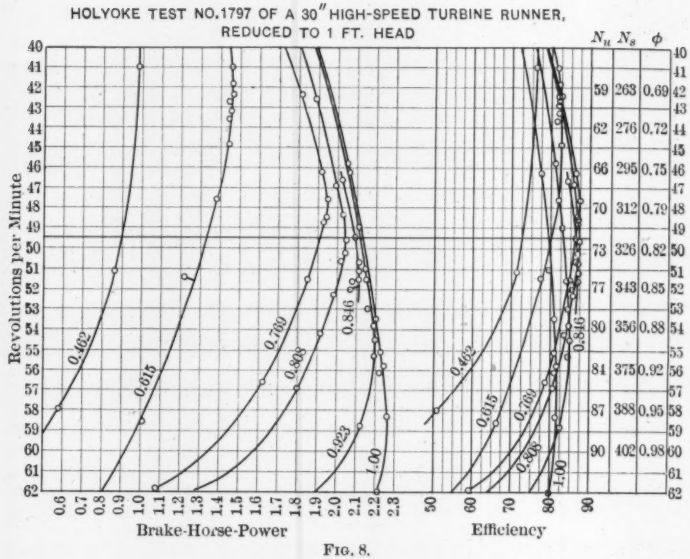

HOLYOKE TEST NO. 1797 OF A 30" HIGH-SPEED TURBINE RUNNER,
REDUCED TO 1 FT. HEAD
N_u N_s ϕ
59 263 0.69
62 276 0.72
66 295 0.75
70 312 0.79
73 326 0.82
77 343 0.85
80 356 0.88
84 375 0.92
87 388 0.95
90 402 0.98
Revolutions per Minute
0.402
0.615
0.789
0.808
0.846
0.923
1.00
Brake-Horse-Power
Efficiency

FIG. 8.

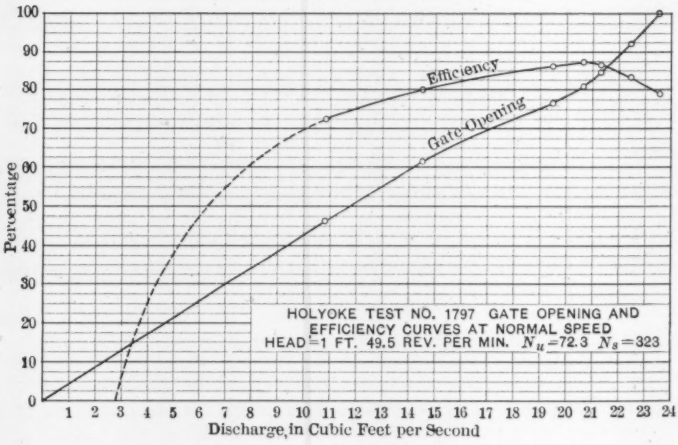

Efficiency
Gate-Opening
Percentage
HOLYOKE TEST NO. 1797 GATE OPENING AND
EFFICIENCY CURVES AT NORMAL SPEED
HEAD=1 FT. 49.5 REV. PER MIN. N_u=72.3 N_s=323
Discharge in Cubic Feet per Second

FIG. 9.

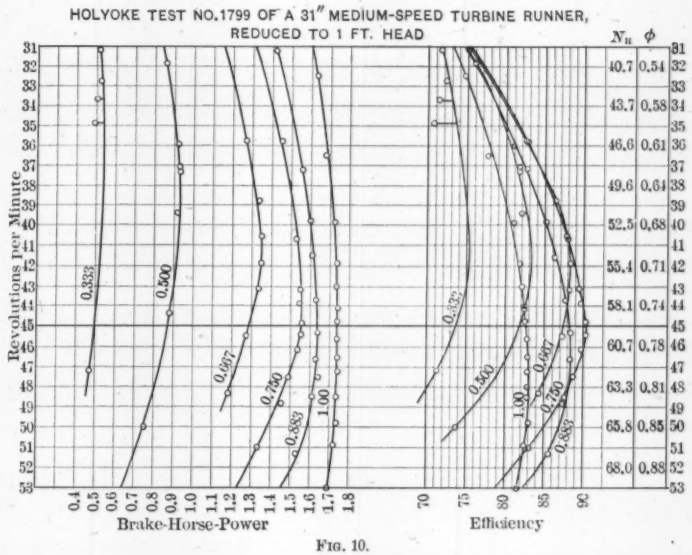

HOLYOKE TEST NO.1799 OF A 31″ MEDIUM-SPEED TURBINE RUNNER,
REDUCED TO 1 FT. HEAD
N_H ϕ
40.7 0.54
43.7 0.58
46.6 0.61
49.6 0.64
52.5 0.68
55.4 0.71
58.1 0.74
60.7 0.78
63.3 0.81
65.8 0.85
68.0 0.88
0.333
0.500
0.667
0.750
0.833
1.00
Revolutions per Minute
Brake-Horse-Power
Efficiency

FIG. 10.

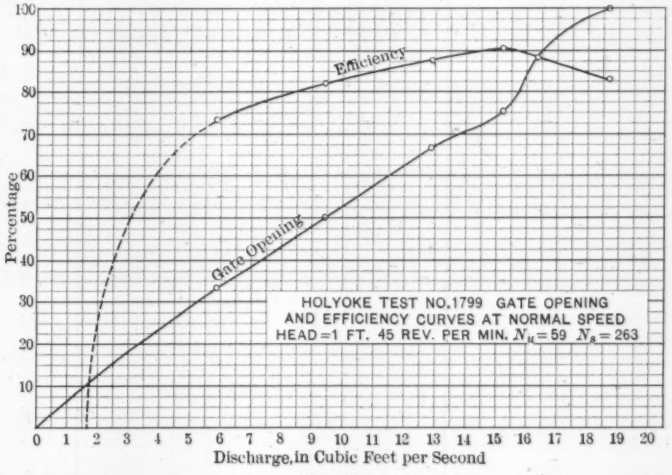

Efficiency
Gate Opening
HOLYOKE TEST NO.1799 GATE OPENING
AND EFFICIENCY CURVES AT NORMAL SPEED
HEAD=1 FT. 45 REV. PER MIN. N_H=59 N_R=263
Percentage
Discharge, in Cubic Feet per Second

FIG. 11.

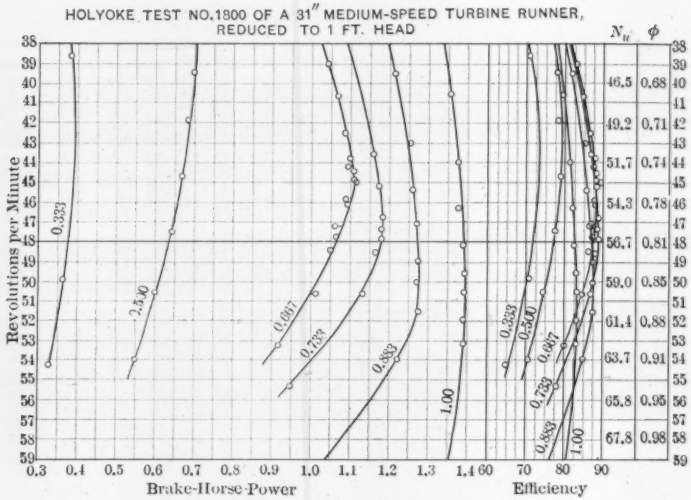

HOLYOKE TEST NO.1800 OF A 31″ MEDIUM-SPEED TURBINE RUNNER,
REDUCED TO 1 FT. HEAD
N_u
ϕ
46.5 0.68
49.2 0.71
51.7 0.74
54.3 0.78
56.7 0.81
59.0 0.85
61.4 0.88
63.7 0.91
65.8 0.95
67.8 0.98
Revolutions per Minute
Brake-Horse-Power
Efficiency
0.333
0.500
0.667
0.733
0.833
1.00

FIG. 12.

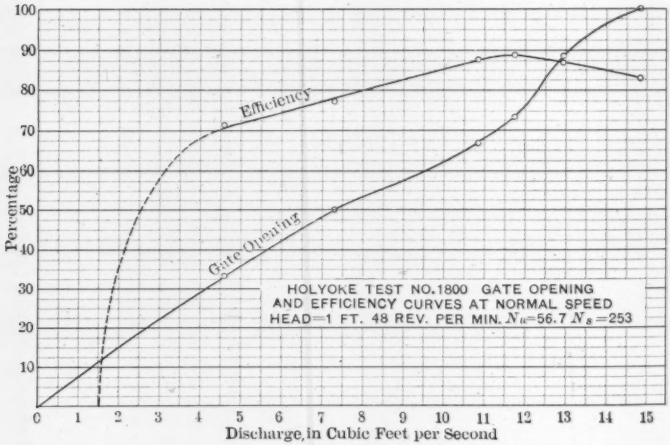

Efficiency
Gate Opening
Percentage
HOLYOKE TEST NO.1800 GATE OPENING
AND EFFICIENCY CURVES AT NORMAL SPEED
HEAD=1 FT. 48 REV. PER MIN. N_u=56.7 N_s=253
Discharge in Cubic Feet per Second

FIG. 13.

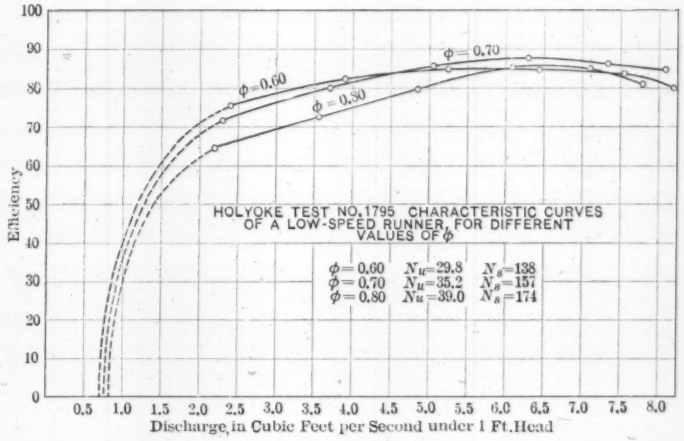

Efficiency
100
90
80
70
60
50
40
30
20
10
0
$\phi = 0.70$
$\phi = 0.60$
$\phi = 0.80$
HOLYOKE TEST NO. 1795 CHARACTERISTIC CURVES OF A LOW-SPEED RUNNER, FOR DIFFERENT VALUES OF ϕ
$\phi = 0.60$ $N_u = 29.8$ $N_s = 138$
$\phi = 0.70$ $N_u = 35.2$ $N_s = 157$
$\phi = 0.80$ $N_u = 39.0$ $N_s = 174$
0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0 4.5 5.0 5.5 6.0 6.5 7.0 7.5 8.0
Discharge, in Cubic Feet per Second under 1 Ft. Head

FIG. 14.

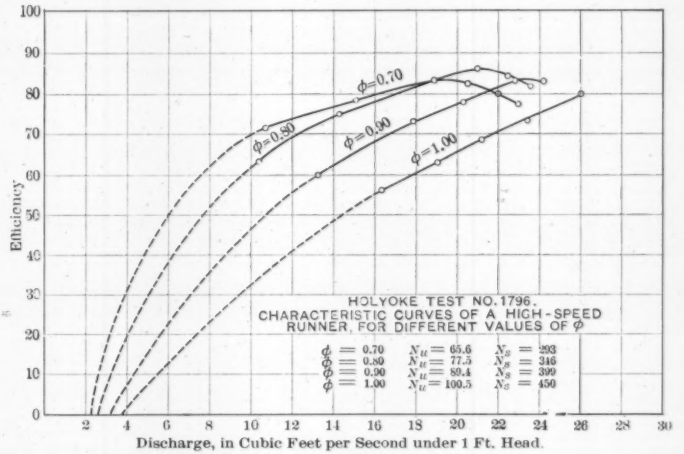

Efficiency
100
90
80
70
60
50
40
30
20
10
0
$\phi = 0.70$
$\phi = 0.80$
$\phi = 0.90$
$\phi = 1.00$
HOLYOKE TEST NO. 1796.
CHARACTERISTIC CURVES OF A HIGH-SPEED RUNNER, FOR DIFFERENT VALUES OF ϕ
$\phi = 0.70$ $N_u = 65.6$ $N_s = 293$
$\phi = 0.80$ $N_u = 77.5$ $N_s = 346$
$\phi = 0.90$ $N_u = 89.4$ $N_s = 399$
$\phi = 1.00$ $N_u = 100.5$ $N_s = 450$
2 4 6 8 10 12 14 16 18 20 22 24 26 28 30
Discharge, in Cubic Feet per Second under 1 Ft. Head.

FIG. 15.

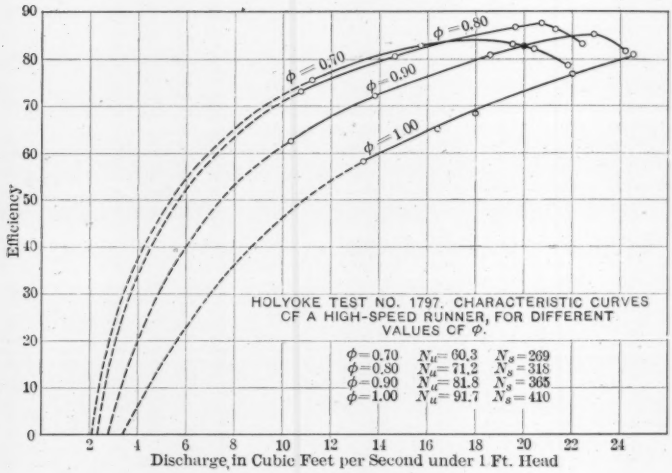

Efficiency
Discharge in Cubic Feet per Second under 1 Ft. Head
$\phi = 0.70$
$\phi = 0.80$
$\phi = 0.90$
$\phi = 1.00$
HOLYOKE TEST NO. 1797. CHARACTERISTIC CURVES OF A HIGH-SPEED RUNNER, FOR DIFFERENT VALUES OF ϕ.
$\phi = 0.70$ $N_H = 60.3$ $N_s = 269$
$\phi = 0.80$ $N_H = 71.2$ $N_s = 318$
$\phi = 0.90$ $N_H = 81.8$ $N_s = 365$
$\phi = 1.00$ $N_H = 91.7$ $N_s = 410$

FIG. 16.

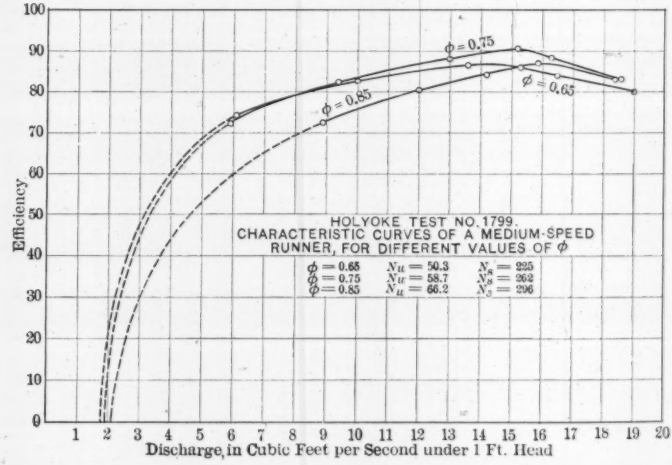

Efficiency
Discharge in Cubic Feet per Second under 1 Ft. Head
$\phi = 0.75$
$\phi = 0.65$
$\phi = 0.85$
HOLYOKE TEST NO. 1799. CHARACTERISTIC CURVES OF A MEDIUM-SPEED RUNNER, FOR DIFFERENT VALUES OF ϕ
$\phi = 0.65$ $N_H = 50.3$ $N_s = 225$
$\phi = 0.75$ $N_H = 58.7$ $N_s = 262$
$\phi = 0.85$ $N_H = 66.2$ $N_s = 296$

FIG. 17.

Efficiency
Discharge in Cubic Feet per Second under 1 Ft. Head
HOLYOKE TEST NO. 1800.
CHARACTERISTIC CURVES OF A MEDIUM-SPEED RUNNER, FOR DIFFERENT VALUES OF φ
φ = 0.70 N_u = 48.6 N_s = 212
φ = 0.80 N_u = 56.0 N_s = 250
φ = 0.90 N_u = 63.0 N_s = 281
φ = 0.80
φ = 0.90
φ = 0.70

FIG. 18.

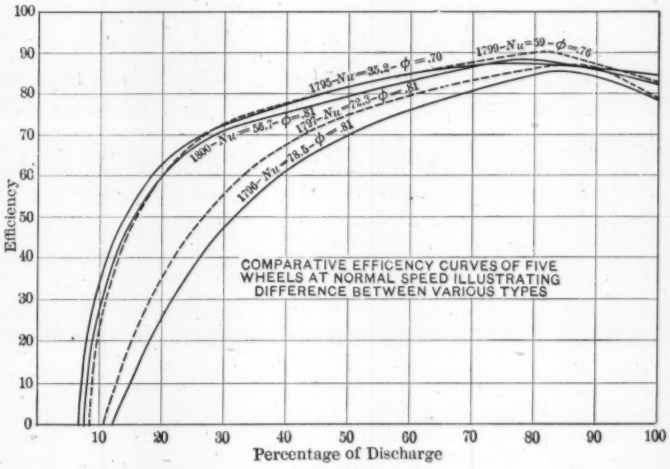

Efficiency
Percentage of Discharge
COMPARATIVE EFFICENCY CURVES OF FIVE WHEELS AT NORMAL SPEED ILLUSTRATING DIFFERENCE BETWEEN VARIOUS TYPES
1795–N_u = 35.2 – φ = .70
1790–N_u = 59 – φ = .76
1800–N_u = 56.7 – φ = .81
1797–N_u = 72.5 – φ = .81
1796–N_u = 73.5 – φ = .81

FIG. 19.

squares of the diameters, and that the speed varies inversely as the diameters, with constant efficiency.

Let D_a = the diameter of A,

D'_a = the diameter of a wheel of homologous design, having a capacity of 1 h.p.,

N'_a = the speed of the latter.

Then

$$D^2_a : D'^2_a = 2.07 : 1$$

$$D_a : D'_a = \sqrt{2.07} : 1$$

also

$$D_a : D'_a = N'_a : 50$$

then

$$\sqrt{2.07} : 1 = N'_a : 50$$

$$N'_a = 50\sqrt{2.07} = 72$$

Similarly $N'_b = 75$.

It is now plain that B is the higher-speed type, because, when both wheels are developing 1 h.p., B runs at 75 rev. per min., whereas A runs at only 72 rev. per min. As will be explained later, these are the "unit speeds" of these two wheels.

It should be clearly understood that this term, "high-speed," thus involves a consideration of power as well as speed. It is not related directly to the peripheral speed, although, as stated before, high peripheral speed is usually an attribute of a high-speed wheel. At the same time, it may also be an attribute of a low-speed wheel, and therefore it is not a proper criterion of speed in the sense in which it is here used. Wheel No. 1800 illustrates this point very well. At the normal speed of this wheel, the coefficient of peripheral speed, $\phi = 0.81$. This value is as high as for any of the other wheels, and yet this wheel is lower in speed than all the others, except No. 1795.

Unit Speed.—The "unit speed," N_u, used throughout this paper, is in all cases the speed, under 1 ft. head, of a wheel of the same design as the one under consideration, and of such size as to develop 1 h.p. at full gate. Other units might be used and serve the same purpose of comparison, but in America these are the most convenient. In Europe, under the metric system, the practice is to reduce to one metric horsepower, under 1 m. head. The result is termed the "specific speed," and is usually designated by the symbol, N_s.

The general equation for N_u is developed as follows:

N = speed of any runner,
P = power of same,
D = diameter of same,
h = head on same,
h_u = unit of head,
P_u = unit of power,
N' = speed of above runner under unit head,
P' = power of above runner under unit head,
D_u = diameter of homologous runner developing unit power under unit head.

Then

$$N' : N = \sqrt{h_u} : \sqrt{h} \qquad N' = N\sqrt{\frac{h_u}{h}} \quad \ldots\ldots (a)$$

$$P' : P = h_u^{\frac{3}{2}} : h^{\frac{3}{2}} \qquad P' = P\left(\frac{h_u}{h}\right)^{\frac{3}{2}} \quad \ldots\ldots (b)$$

$$P' : P_u = D^2 : D_u^{\,2} \qquad D_u = D\sqrt{\frac{P_u}{P'}} \quad \ldots\ldots (c)$$

$$\frac{N'}{N_u} = \frac{D_u}{D} \quad \ldots\ldots (d)$$

Substituting Equations a, b, and c in Equation d.

$$\frac{N\sqrt{\frac{h_u}{h}}}{N_u} = \frac{D\sqrt{\frac{P_u}{P\left(\frac{h_u}{h}\right)^{\frac{3}{2}}}}}{D} = \sqrt{\frac{P_u}{P\left(\frac{h_u}{h}\right)^{\frac{3}{2}}}}$$

Simplifying,

$$N_u = \frac{N \times \sqrt{P} \times h_u^{\frac{5}{4}}}{\sqrt{P_u} \times h^{\frac{5}{4}}} \quad \ldots\ldots (1)$$

With $P_u = 1$ h.p. and $h_u = 1$ ft. this becomes

$$N_u = \frac{N \times \sqrt{P}}{h^{\frac{5}{4}}} \quad \ldots\ldots (2)$$

Given the power and speed of a wheel under any head, the unit speed may be determined by substituting in Equation 2. It is not

necessary to go through the process of double reduction as was done in comparing wheels A and B. For example, take wheel A, just considered.

Here

$$h = 100 \qquad P = 2\,080 \qquad N = 500$$

Substituting in Equation 2,

$$N_u = \frac{500 \times \sqrt{2\,080}}{100^{\frac{5}{4}}} = 72$$

which checks the former computation. Under the metric system, $P_u = 0.98$ h.p. and $h_u = 3.28$ ft. Substituting in Equation 1.

$$N_s = \frac{N \times \sqrt{P} \times 3.28^{\frac{5}{4}}}{\sqrt{0.98} \times h^{\frac{5}{4}}} = 4.46 \times \frac{N \times \sqrt{P}}{h^{\frac{5}{4}}} \qquad (3)$$

or

$$N_s = 4.46\ N_u.$$

Hence, to transform a unit speed from the English to the metric system, multiply N_u by the coefficient, 4.46.

This method of classifying wheels according to their unit speeds is one of great convenience. Suppose, for example, that it is desired to equip a plant with units using runners having a maximum capacity of 1 000 h.p. each, at 200 rev. per min., under 40 ft. head. From Equation 2, $N_u = 63.3$, and therefore only wheels approximating this unit speed need be considered.

Again, suppose that the sizes of the units and the speed for a prospective plant under 100 ft. head have not been determined. It is desired to know what results could be obtained with a certain runner, the characteristics of which are well suited to the proposed service. The unit speed of this runner is 70. Substituting in Equation 2,

$$70 = \frac{N \times \sqrt{P}}{100^{\frac{5}{4}}} \qquad N \times \sqrt{P} = 22\,136.$$

This type of wheel will give any combination of power and speed, whatsoever, that will satisfy this equation. With the inverted scale of a duplex slide-rule, innumerable combinations of power and speed may be read off as rapidly as one can move the runner. This is a great convenience in preliminary calculations. In the above case, for

instance, any of the following combinations of power and speed can be produced, depending on the size of the runner used.

1 000	h.p.	at	700	rev. per min.
1 360	"	at	600	"
1 960	"	at	500	"
3 060	"	at	400	"
5 450	"	at	300	"
12 200	"	at	200	"
49 000	"	at	100	"

Classification According to Unit Speeds.—Some method of classifying turbine runners on the basis of the duty which they are able to perform is essential to an intelligent study of turbine characteristics. It is obviously futile to compare the relative merits of two wheels which cannot accomplish the same work. They are in different classes, and, as such, might naturally be expected to possess different characteristics.

Since the unit speed is a measure of complete performance, both as to speed and power, it supplies an admirable basis for the desired classification. In this it should be noted that such a classification is independent of the details of design. The angles of the vanes, general proportions, peripheral speed, etc., may all be different, and yet the wheels are classified as of the same type if they have the same unit speed, because they, and they alone, are capable of doing the same work.

In Europe it is the practice to classify runners according to their coefficients of peripheral speed. A "low-speed" runner has a value of ϕ equal to about 0.6, a "normal-speed" runner, about 0.7 and a "high-speed" runner, about 0.8. These distinctions, however, embody the element of capacity as well as speed, although in an indirect and rather indefinite way. This is because a value of $\phi = 0.6$ is not ordinarily used, except with wheels of low capacity, 0.7 with those of normal capacity, and 0.8 with those of high capacity. The distinguishing element of capacity is really there, although its influence is not definitely measured. Such a method, of course, cannot be as exact as that based on unit speeds. In the latter case, both capacity and speed are definite mathematical factors.

Influence of ϕ Upon the Characteristic Curves.—The extent to which changes in the value of ϕ at normal speed affect the efficiency curves of turbine runners is difficult to determine, because, with wheels of rational design, the value of ϕ, as just stated, keeps step with the variation in the capacity of the wheel. It is, of course, a principle of economical design that a runner shall be made no larger in diameter than necessary. For a given speed, the lower the value of ϕ the smaller the wheel, and hence a low value of ϕ is suitable to a low-capacity wheel, and a high value to a wheel of large capacity. It may be generally stated as a principle of rational design that ϕ should always be made as low as possible, and that it should be forced up to high values only in order to get additional capacity.

Wheel No. 1800 was an experimental runner designed contrary to this principle for purposes of investigation. It is of irrational design in that it is a wheel of comparatively low capacity with a high value of ϕ. The value of ϕ for a wheel of this capacity need not be over 0.7 at normal speed, whereas it was 0.81, according to test.

The results obtained with this wheel appear to indicate that, with wheels of similar capacity, the value of ϕ at normal speed has no very important effect on the characteristic curves, although a high value of ϕ does show some tendency to pull down the efficiency. Fig. 19 shows that the three wheels of medium and low capacity gave similar results at normal speed, although the values of ϕ were 0.70, 0.76, and 0.81, respectively. The three wheels, however, which had the same value of ϕ, namely, 0.81, but differed considerably in capacity, ranging from medium power to the extreme limit of high power, gave widely different curves. All of which seems to indicate that forced capacity is the chief factor which causes the variation.

A comparison of Figs. 14, 17, and 18 shows a further similarity between the three wheels having different values of ϕ. The variation in efficiency at part gates, due to a variation of 0.1 in ϕ both above and below its normal value, is quite similar in all three cases. In fact, what difference there is between the three wheels is in favor of the high value, $\phi = 0.81$. This wheel shows less drop in efficiency at part gates, due to an increase in speed, than either of the others.

In view of this evidence, and for lack of any experimental proof to the contrary, the writer believes it safe to say that the efficiency is not a function of ϕ, to any considerable extent.

Principal Difference Between Low-Speed and High-Speed Wheels.—Before proceeding further, it may be well to state that the deductions made from these tests should not be given too strict an application. Results necessarily vary with changes in design, and, while the results obtained with these wheels are of a nature broadly representative of other wheels of similar capacity, nevertheless, considerable variation in individual cases must be expected.

The most significant fact to be observed, in connection with the efficiency-speed curves of these wheels at various gates (Figs. 4, 6, 8, 10, and 12), is that all of them peak at different speeds. The most efficient speed of the wheel varies with the gate opening, increasing as the latter increases. This is true of all reaction turbines, whether of low or high speed, but not in the same degree. In the case of high-speed wheels, the difference between the best low-gate speed and the best full-gate speed is much greater than in the case of medium-speed and low-speed wheels. Table 6 shows the comparison between 0.4 gate and full gate for all the wheels under 1 ft. head, listed in the order of their capacity.

TABLE 6.

Wheel No.	1795	1800	1799	1797	1796
Most efficient full-gate speed	40	52	48	58	64
Most efficient 0.4-gate speed	31	41	38	35	35
Percentage of drop in speed	22.5	21.1	20.8	39.6	45.3
N_u	35.2	56.7	59.0	72.3	78.5

It may be observed from Table 6 that the drop for low-speed wheel No. 1795 and for the two medium-speed wheels, Nos. 1799 and 1800, is about the same, whereas it is nearly twice as great for the high-speed wheel No. 1797, and even greater for the extremely high-speed wheel No. 1796. This drop does not seem to vary much with the capacity of the wheel for values of N_u below 60, approximately. Above $N_u = 60$, or thereabouts, the drop increases rapidly as N_u increases.

When operating at constant speed, the inevitable result of this variation in the most efficient speed of the wheel is a sacrifice of efficiency at all gates except one. For example, if a speed is selected which will give the highest efficiency which it is possible to obtain at 0.75 gate, that speed will be too high to give the best obtainable efficiency at the lower gates and too low to give the best results at the higher gates. The best speed at which to run a wheel, therefore,

depends on which gate opening is the most important as regards efficiency. The best speed is the one which strikes the peak of the efficiency curve for that gate.

Naturally, the less variation there is in the best speed from low gate to full gate the less sacrifice of efficiency there will be when the wheel is run at constant speed. It has just been shown that this variation is much less with wheels of low and moderate unit speeds than with those of high unit speed, and therein lies the most important difference between these several types.

As an illustration of this point, compare low-speed wheel No. 1795 with high-speed wheel No. 1796 (Figs. 4 and 6). The maximum efficiency of the former at 0.444 gate is 82%, occurring at a speed of 34.5 rev. per min. The efficiency at the normal speed of 40 rev. per min., however, is 80%, and hence 2% has been sacrificed at this gate to the requirement of constant speed.

In the case of No. 1796, the maximum efficiency at 0.462 gate is 74.8%, occurring at 37.8 rev. per min. At normal speed of 53 rev. per min. the efficiency is only 62.5% and about 12% has been sacrificed to the requirement of constant speed. In addition to this, the peak of the curve of No. 1795 is about 7% higher than that of No. 1796, and hence the total difference in favor of the low-speed wheel at this gate is about 17 per cent. Fig. 19 shows graphically what a wide difference exists between the efficiency curves of medium-speed and high-speed wheels. The relatively poor showing of the two high-speed wheels, Nos. 1796 and 1797, is due to the two characteristic deficiencies just alluded to, namely, rapid drop of the efficiency-speed curves at part gate, and relatively lower maximum efficiencies at all gates. The latter condition is theoretically inevitable. It is impossible to discharge a large quantity of water through a wheel of given size as efficiently as a moderate quantity.

Another peculiarity which is distinctly characteristic of high-speed wheels is that the power at the high gates increases as the speed increases beyond normal. By over-gating the wheel it is possible to hold up a substantial increase of power with good efficiency at extremely high values of ϕ. For example, wheel No. 1796 (Fig. 6), at 1.077 gate, increases constantly in power and efficiency until ϕ has reached a value of 1.00. No. 1797 (Fig. 8), at 1.00 gate, increases until $\phi = 0.97$.

This feature of high-speed wheels is a valuable one, because it fits them especially for the only work for which they are at all adapted, namely, low heads. Low-head plants are almost invariably subject to relatively large reductions of head, due to back-water in the tail-race. If the head drops very much, the value of ϕ necessary to maintain constant speed will run up very high. A wheel which gives more power at a high value of ϕ than it does at the normal value, is a great advantage in such a case, because it increases the capacity of the turbine by just that much at a time when the power is perilously deficient on account of the reduced head. At such a time, it is necessary to open the turbine gates farther than would be advisable under the normal head. Under the latter, such over-gating would be wasteful, because the increase in power is very small and not at all porportionate to the increase in discharge. With some wheels, the power is even less than it was at the previous gate, on account of excessive drop in efficiency. Under the reduced head, however, with high values of ϕ, the increase in power is quite considerable. With No. 1796 (Fig. 6), for instance, the increase in power from 1.00 gate to 1.077 gate, at normal speed, is practically nothing, due to a 4% drop in efficiency. At $\phi = 1.00$, however, the increase in power is about 20%, and the efficiency is about 6% better. The over-gate, therefore, should be used only under a reduced head.

Reference to Figs. 4, 10, and 12 will show that the wheels of lower unit speed do not possess this characteristic. In every case, the high-gate curves fall off at the high values of ϕ.

Considerations Affecting the Choice of Wheels for Low Heads.— It is futile, of course, to attempt to define within fixed limits the heads which should be included in this classification. Some plants under moderately high heads operate under conditions very similar to those which are distinctly characteristic of the typical low-head plant. Others under low heads possess some of the characteristics common to high heads. If any useful deductions are to be drawn from such a study, they must be based on a consideration of typical conditions, with a liberal allowance for the inevitable exceptions, which require special treatment.

A typical low-head plant may be illustrated by an installation where the head is created by building a dam across a large river flowing through a comparatively flat country. In such cases, it is im-

practicable to build a high dam, because the water cannot be confined behind it.

Having, probably, little storage area, other than the bed of the stream, such a power cannot usually be developed much beyond the minimum flow of the stream. If it were possible to impound the water which is not used during light loads for use during peak loads, then the part-gate efficiency of the turbines for such a plant would be of importance; but, usually, the water which is not used during light loads goes over the dam, and therefore the efficiency of the turbines at such times is relatively unimportant. Under such conditions, the poor part-gate efficiency of high-speed wheels is of no importance.

The important considerations are high efficiency at normal gate, as affecting the normal quantity of power which can be developed from the available flow; high speed, as affecting the cost of the electrical machinery and, to a limited extent, the cost of the turbines; high power at constant speed under reduced heads due to back-water, which is the inevitable coincident of flood conditions.

In respect to the first consideration, the high-speed wheel is somewhat deficient, depending on the extent to which the wheel is pushed for speed. Taking the efficiency of No. 1799, 90.43%, as the standard for a wheel of medium speed, the high-speed wheel No. 1797, yielding 87.39%, suffers about 3% by comparison. Wheel No. 1796, which is a more extreme type than No. 1797, gave 86.15%, or about 4¼% less.

Another point to be noted is that the maximum efficiency of high-speed wheels occurs at a higher gate opening than that of lower-speed wheels. With the latter it usually occurs at from 0.70 to 0.75 gate, whereas with the former it is usually from 0.80 to 0.85 gate. In order to provide for the customary overload of 25% with high-speed wheels, it is necessary that they shall carry the normal load at a lower gate than the most efficient one. This causes a further loss of efficiency.

In respect to the second and third considerations, the high-speed wheel has an overwhelming advantage, and is undoubtedly the best wheel for typical low-head service. Table 7 gives a comparison of the results which could be accomplished by the three types of runner, Nos. 1796, 1797, and 1799, respectively, if applied to a plant having a maximum capacity of 500 h.p. per runner under 20 ft. head. Low head is assumed to be 13 ft.

TABLE 7.

Wheel. No.	20 Feet.			13 Feet.			N_u
	Horse-power.	Revolutions per minute.	Efficiency.	Horse-power.	Revolutions per minute.	Efficiency.	
1796	500	149	82.0	279	149	80.0	78.5
1797	500	136	83.0	277	136	80.5	72.3
1799	500	112	82.7	246	112	79.5	59.0

Runners for Intermediate Heads.—It is as the head increases and back-water conditions become less important, and as the speed of the ordinary runner, increasing with the head, more nearly approaches an economical value, that the use of the high-speed runner becomes more questionable. As these conditions change, the considerations which favor high-speed wheels gradually disappear, and others of more importance become dominant.

As the head increases the storage capacity usually increases, and, with the capacity to impound water for peak loads, the part-load economy of the turbines becomes highly important, and the medium-speed wheel with its high efficiency is clearly indicated. The higher the head the more valuable a cubic foot of water becomes, and the more important it is to save it. High speed becomes a subordinate consideration. If the additional power which can be developed by the lower-speed wheels with their high efficiency is worth more than the interest on the saving in electrical apparatus which would result from the use of high-speed wheels, then the former are certainly the more economical proposition. This principle, of course, is fundamental and commonly recognized, but is not always intelligently applied, for lack of specific knowledge regarding the disadvantages incidental to high speed. If purchasers were as well informed as they ought to be, as to the deficiencies of high-speed runners, there would be fewer of them in use at the present time. It is hoped that this paper may supply some welcome information.

Table 8 shows comparative efficiencies at various gates for medium-speed wheel No. 1799 and high-speed wheel No. 1796. In view of these figures, a very weighty reason would be required to justify the use of No. 1796 instead of No. 1799, if efficiency is of any importance.

High-Head Wheels.—Runners for high heads pass through all the intermediate stages of design between the mixed-flow wheel and the

purely inward-flow or so-called Francis type, according to the head available and the power developed. As the head increases the unit speed tends to decrease, and the design approaches that of the low-capacity Francis type (No. 1795). Although the designs of these wheels vary considerably, the characteristic curves vary but slightly from the best results of medium-speed wheels.

TABLE 8.

Wheel No.	Gate Opening.									
	0.30	0.40	0.50	0.60	0.70	0.75	0.80	0.85	0.90	1.00
1799	68.5	76.5	82.0	86.0	88.0	90.4	90.0	89.2	88.0	83.0
1796	41.0	55.5	66.0	73.5	79.5	82.0	84.5	86.2	85.0	82.0

The efficiency curves of wheel No. 1795 (Fig. 5) at normal speed, with $N_u = 35.2$, and wheel No. 1799 (Fig. 11), with $N_u = 59$, are very similar. On account of the discrepancy between the peaks of these two curves, the part-gate efficiencies are about the same (Fig. 19). Otherwise, the latter would be a trifle higher for the low-speed wheel. The general tendency is for the efficiency, particularly at low gates, to improve as N_u decreases.

The reason wheel No. 1795 did not show as high maximum efficiency as No. 1799 is that, being a low-speed wheel, it developed much less power under test, and hence the loss, due to mechanical friction, was a much greater percentage of the total power developed than was the case with No. 1799. Wheel No. 1795, operating under a head of several hundred feet, particularly in large diameters, would undoubtedly develop 90% efficiency or better. The effect of mechanical friction, which has been discussed at length, is especially detrimental to the efficiency of a low-power wheel such as this, when tested at Holyoke. No high-head wheel can by any possibility show as high efficiency in the Holyoke flume as it would in regular operation, assuming, of course, that in the latter case it is properly installed.

DISCUSSION

LEWIS F. MOODY, ESQ.* (by letter).—This may be considered the most noteworthy contribution on the subject since the late R. H. Thurston, M. Am. Soc. C. E., presented his paper on "The Systematic Testing of Turbine Water-Wheels in the United States." This paper was presented before the American Society of Mechanical Engineers in 1887.† The turbine tests reported by Professor Thurston, like those reported by Mr. Larner, were made at the Holyoke flume, and were notable for the efficiencies developed, the highest which had been obtained in reliable tests up to that time. The tests were made in 1883.‡ Mr. Moody.

The wheel tested by Professor Thurston was a 36-in. R. H. "Hercules" turbine, built by the Holyoke Machine Company, and the maximum efficiency, on each of the three tests made by him, was 86.94%, 86.18%, and 87.08%, respectively (Tables IV, V, and VI). The specific speed corresponding to the point of maximum efficiency is 196, in the metric system.

Since the date of Professor Thurston's paper, great progress has been made in water-wheel design, both in America and in Europe. Mr. Larner's paper, and the remarkable results reported by him, indicate the recent advances which have been made in raising both the efficiency and speed of turbines. Mr. Larner's wheels have shown specific speeds as high as 450 (in the metric) or 100, in the foot-pound system, with an efficiency of practically 80 per cent.

As efficiency and specific speed are the most important issues raised in Mr. Larner's paper, the writer has endeavored to show how they are related, and also how various turbines compare in regard to these two factors, by constructing a curve, Plate XXXII, with specific speed for the scale of abscissas, and efficiencies for the ordinates, using the results of all available tests showing the highest known efficiencies. This curve furnishes a basis for comparing the performance of any turbine with the best previous results shown by turbines of the same specific speed.

In plotting it, the speed, head, and delivered horse-power at the point of highest efficiency of a given wheel have been used in calculating the specific speed. That is, a wheel, when running at a gate opening in excess of that giving maximum efficiency, and at a speed in excess of the normal speed, may develop a much higher specific speed than that plotted; but, for the purpose of comparison, the specific speed corresponding to the run on which the efficiency was developed, has been used in plotting the point. On wheels developing unusually high

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† *Transactions*, Am. Soc. Mech. Engrs., Vol. VIII, p. 359.

‡ Bodmer, "Hydraulic Motors," pp. 401, 481–490.

Mr. Moody. speed, additional points have, in some cases, been plotted, giving the efficiencies corresponding to higher specific speeds for the same wheel, and showing by a short curve how the efficiency drops on a given wheel as the specific speed is increased beyond the normal.

Thus, points 24 and 27 on Plate XXXII both correspond to the same wheel, a 56-in. Samson turbine, point 24 corresponding with the maximum efficiency shown by the test, and point 27 showing that with a much greater specific speed the efficiency dropped somewhat but not greatly in this case (this curve is indicated by a dash and two dots). Other tests of various Samson turbines also give points lying on this same curve, indicating similarity of design.

In the same way, a Holyoke test (No. 1778), reported by the Allis-Chalmers Company, has been plotted at several points, giving the curve shown by a fine line; and the test of a Swedish turbine (built by the Karlstad mek. Verkstad, Filiale in Kristinehamn, Sweden),* is given by a dotted line, *H-I*.

It will also be noticed that the highest curve given (a solid, heavy line) passes through two points of Mr. Larner's wheel No. 1796. This indicates that, while no turbine is known (to the writer, at least) which exceeds in efficiency the results shown by this test, No. 1796, at the speeds corresponding to it, still, the high-speed point ($N_s = 446$) was not obtained at the point of maximum efficiency of this wheel; and it would seem probable that a wheel could be designed for this particular specific speed which would exceed this value of the efficiency. (The specific speeds, N_s, are in the metric system throughout, in order to adhere to an international standard.) The equivalent "unit speeds," to use Mr. Larner's term, or "type characteristics," Mr. Zowski's term—both meaning specific speed converted to the foot-pound system of measures—are given at the top of the diagram.†

In order to show the recent progress of water-wheel design, and to compare American and foreign achievements in this branch of engineering activity, a curve constructed by Messrs. V. Graf and D. Thoma,‡ has been reproduced by dots and dashes. This curve was at that time of the same nature, and was intended to show the same things as the writer's full-line curve on Plate XXXII. That is, Graf and Thoma intended their curve to represent the best known results, and, up to a specific speed of 200, it represented very fairly the state of the art at that time. Above that speed, however, their curve fell considerably below many results which had been obtained. The reason for this was that many German engineers did not then, and do not now, accept Holyoke tests as correct, and are inclined to ignore all results obtained in America. There were many American tests considerably exceeding the curve of

* *Zeitschrift für das gesamte Turbinenwesen*, Heft 3, V Jahrgang, Jan. 30th, 1908.

† See article by S. J. Zowski, *Engineering News*, Jan. 28th, 1909.

‡ Published in the *Zeitschrift des Vereines deutscher Ingenieure*, June 29th, 1907.

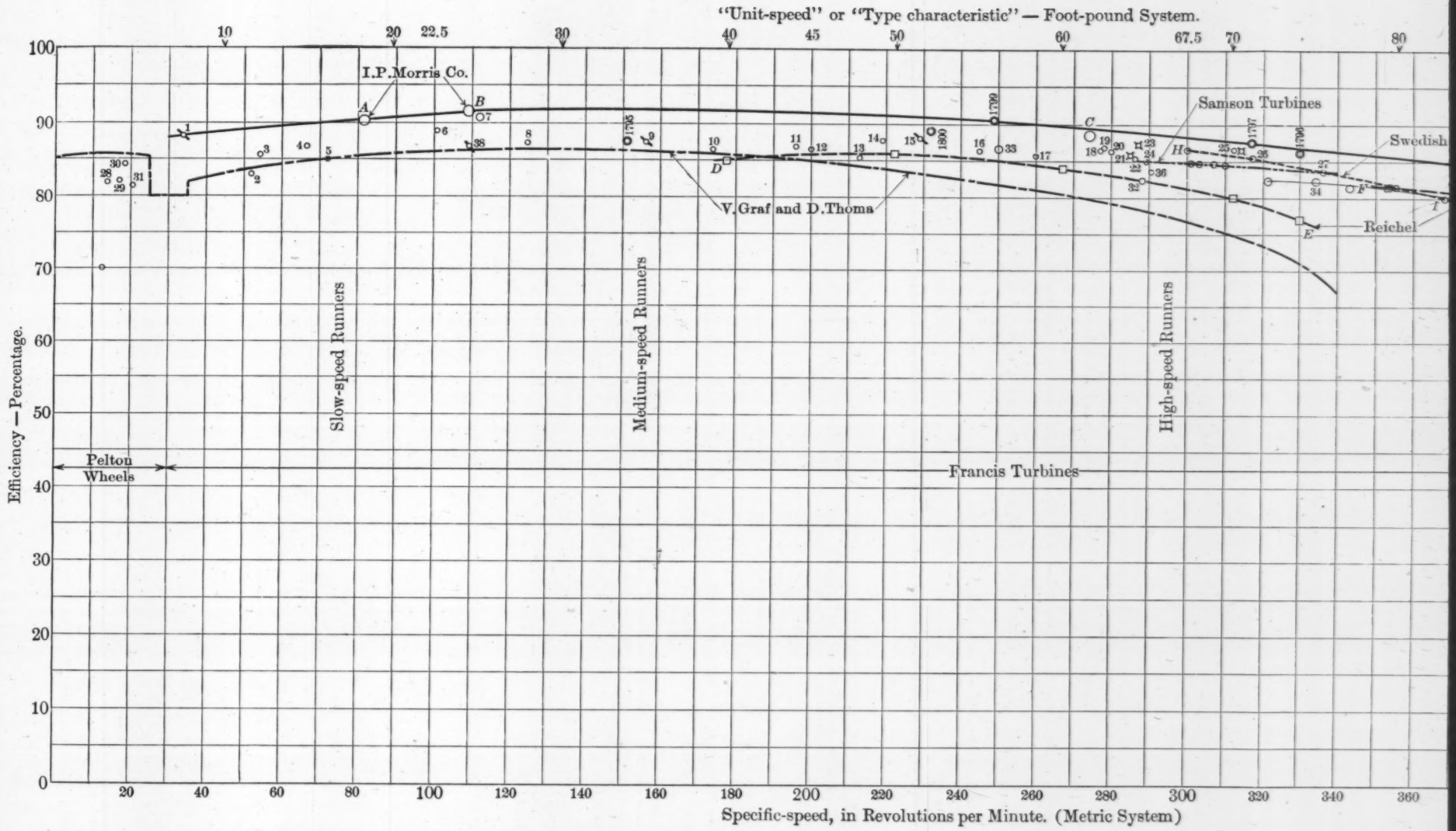
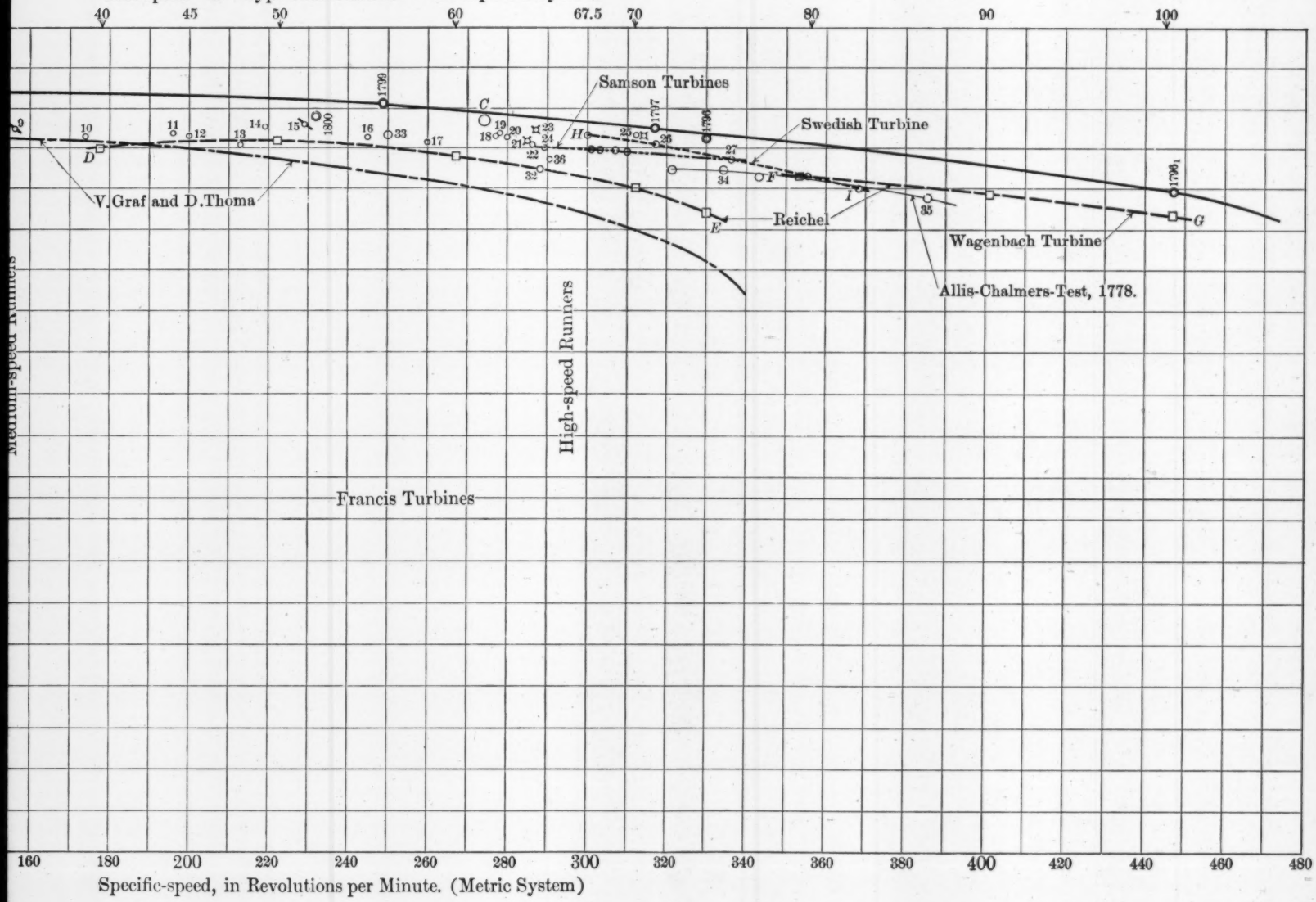

"Unit-speed" or "Type characteristic" — Foot-pound System.
10
20
22.5
30
40
45
50
60
67.5
70
80
100
90
80
70
60
50
40
30
20
10
0
I.P.Morris Co.
A
B
C
D
E
F
H
Samson Turbines
Swedish
Reichel
V.Graf and D.Thoma
Slow-speed Runners
Medium-speed Runners
High-speed Runners
Pelton Wheels
Francis Turbines
Efficiency — Percentage.
20
40
60
80
100
120
140
160
180
200
220
240
260
280
300
320
340
360
Specific-speed, in Revolutions per Minute. (Metric System)

PLATE XXXII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXVI, No. 1138.
MOODY ON
MODERN HYDRAULIC TURBINES.



Mr. Moody.

Graf and Thoma in efficiency and speed; but, as has been said, very few available results have approached the new curve plotted by the writer on the basis of Mr. Larner's tests and some other recent results.

Since the international aspect of the question under discussion naturally presents itself, the curves furnished by Professor Ernst Reichel,* have been indicated by curves of dashes at *D-E* and *F-G*. As stated in the article mentioned, the curve, *D-E*, represents present German accomplishments, and shows the progress that has been made in Germany since the Graf and Thoma article was written. The second curve, *F-G*, is from tests of an experimental turbine designed by Wagenbach, and indicates the highest specific speed thus far attained in Germany. The efficiencies are below those given in Mr. Larner's paper by from 3 to 4 per cent.

In constructing the curve for maximum attained efficiencies, the writer, besides plotting all the tests known to him in printed form, wrote to eight of the leading turbine manufacturers in the United States for their most recent and highest results; and, although he has not yet heard from all of them, he believes that the data thus collected represent very fairly the best American practice to-day. With only two exceptions, the manufacturers have responded most liberally to this request, and, while the writer is not able to assume responsibility for the accuracy of the data, he has been furnished with signed reports of the tests, and believes them to be authentic, with few if any exceptions.

As many of these tests were made at Holyoke, and as the question of the accuracy of Holyoke tests has already been raised, the writer takes this occasion to state that he has had the opportunity to investigate fully the methods used at the Holyoke flume, and is convinced of the substantial accuracy of the Holyoke results. During the test of a 33-in. turbine built by the I. P. Morris Company, at which the writer was present as the representative of that company, he was able to check up the constants of the flume, measuring the dimensions of the brake, etc., checking the elevation of the weir crest with a level, and checking the head on the weir with an independent glass gauge during the test.

No sources of systematic error were found; and the results are believed to be as accurate as the Francis weir formula, within the limits of its use at Holyoke.

The specific speed-efficiency curve, as plotted, passes through the following points:

Point 1 is from a test of a 2 000-h.p. horizontal, spiral-casing turbine with double runner, operating under a head of 122 m., at Rauris-Kitzloch (Salzburg), Austria. It was built by Escher Wyss and Company, of Zürich, for the Aluminium-Industrie A.-G. Neuhausen, and developed a maximum efficiency of 88.2% with a specific speed of 34.

* Recently published in *Engineering News*, September 9th, 1909, p. 286.

Mr. Moody. The data for this and other tests of Escher Wyss wheels are furnished by the bulletin published by the Escher Wyss Company (1908).

Points *A* and *B* are from tests of two wheels built by the I. P. Morris Company, of Philadelphia, the guides and runners of which were designed by the writer, under the direction of Mr. W. M. White, Hydraulic Engineer of the company. Point *A* represents the 750-h.p. turbines, built for J. G. White and Company, and installed at the Comerio Falls Plant, Porto Rico, of the Porto Rico Light and Power Company. These turbines are of the horizontal, volute-casing type, with single runner and single draft-tube. They have movable guide-vanes, or "wicket-gates," with outside operating mechanism; and the runners are of bronze. When tested in place, the output being measured electrically and the water by a Francis weir, they showed an efficiency of 90.3% when developing 755 h.p. under a head of 182.75 ft. The speed was 450 rev. per min.

Point *B* is from the acceptance test of unit No. 12 at power-house No. 3 of the Niagara Falls Hydraulic Power and Manufacturing Company. The test was made in place, under operating conditions, by the engineers of that company and of the I. P. Morris Company, and was very carefully conducted. The water measurement was by weir, using the Francis formula. The turbines are of 10 000 h.p., of the horizontal, volute-casing type with double runners and two draft-tubes per unit. They have movable guide-vanes and bronze runners. The test showed that when delivering 8 934 h.p. under a head of 214.3 ft., at a speed of 299.9 rev. per min., the efficiency was 91.7 per cent. Mr. White states in a letter:

"I have no doubt but that the 10 000-h.p. units are the most efficient units in operation to-day."

Point 7, just below the curve, is from a test furnished by the Pelton Water Wheel Company on a turbine built by that company for the Black Hills Traction Company at Spearfish, S. Dak. The test showed a maximum efficiency of 90.8%, as read from the curve. The wheel is of the "Pelton-Francis" type, with volute casing and single runner, and was tested in place by the engineers of the companies.

Point *C* is from the Holyoke test of the 33-in., I. P. Morris Company wheel previously mentioned. This wheel showed an efficiency of 88.49%, with 17.51-ft. head, 184.57 rev. per min., and 142.18 h.p., the corresponding specific speed being 274.

Other points on the curve are from Mr. Larner's paper, and are indicated by the numbers used by him.

The only other turbine known to the writer which, in efficiency, approaches very closely the values given on the curve, is a French wheel built by the *Société des Établissements Singrun*, at Epinal, for the French Government, the efficiency of which is reported to be 90.4 per

cent. The data for calculating the specific speed have not been ascertained. Mr. Moody.

Many excellent results not falling on the curve, but showing very creditable efficiencies and speeds, are shown in Table 9.

TABLE 9.

Number of point	Turbine.	Builder.	Place of test.	Efficiency. Percentage.	Specific speed, N_s.
20	Hercules 36-in.	Holyoke Machine Co.	Holyoke Flume.	86.17	280
21	" 36-in.	" " "	" "	86.51	285
23	" 30-in.	" " "	" "	87.07	287
25	" 39-in.	" " "	" "	86.50	314
24	Samson 56-in.	James Leffel & Co.	" "	85.01	289
27				83.58	336
36	" 50-in.	" " " "	" "	83.59	291
..	" 45-in.	" " " "	" "	84.56	310
..	" 40-in.	" " " "	" "	84.72	307
..	" 35-in.	" " " "	" "	84.78	303
5	Pelton-Francis	Pelton Water Wheel Co.	In place, Sultepec	85.0	72.5
10	" "	" " " "	In place, Schaghticoke	86.5	174
33	Type E-30-in.	Allis-Chalmers Co.	Holyoke	86.51	250
..				82.31	321
34	Type F-30-in.	" " "	"	82.5	334
..				81.5	343
35				78.89	385
8	10 500-h.p. Shawinigan	I. P. Morris Co.	In place	87.2	125
3	Samson Niagara Type	James Leffel & Co.	" "	85.6	54.5
4	6 100-h.p. at Hamilton Cat. Co.	J. M. Voith, Heidenheim.	" "	86.8	66.5*
9	At Rheinfelden	Escher Wyss & Co.	" "	87.4	156.5
15	At Manegg-Zürcher Papierfabrik	" " " "	" "	88.0	229
38	For A. Giraud & Cie., Lyon	" " " "	" "	86.8	109.4
32	Experimental at Sagan	Designed by Dr. Camerer.	In Germany, Experimental Flume	82.3	288
13	Swain	Swain Turbine & Mfg. Co.	Holyoke	85.39	213†
17	McCormick 39-in.	S. Morgan Smith Co.	"	85.78	260†
19	McCormick 51-in.	J. & W. Jolly	"	86.84	278†
18	McCormick 33-in.	S. Morgan Smith Co.	"	86.3	277†

NOTE.—Point 11, at 196 specific speed, is Professor Thurston's test previously referred to.
* From "Hydro-electric Developments and Engineering," by F. Koester.
† From "Water Power Engineering," By D. W. Mead, M. Am. Soc. C. E.

In conclusion, the writer wishes again to express his appreciation of Mr. Larner's valuable paper; and to extend thanks, for their courtesy in furnishing information, to the following: Mr. H. Birchard Taylor, Assistant Hydraulic Engineer, I. P. Morris Company; Mr. J. E. Strothman, Assistant Manager, Pumping Engine and Hydraulic Turbine Department, Allis-Chalmers Company; The Holyoke Machine Company, Mr. W. W. White, Manager; The Pelton Water Wheel Company, Mr. J. V. Kunze, Manager; James Leffel and Company; and The Platt Iron Works Company, Mr. John Sturgess, Western Representative. It is regretted that lack of time has prevented the writer

Mr. Moody. from awaiting the data which are being sent by the Platt Iron Works Company.

Mr. Parker. JOHN C. PARKER, ASSOC. M. AM. SOC. C. E.—In this paper Mr. Larner has contributed so much that is of signal technical interest that one may lose sight of some of his general remarks. The speaker would like to call attention to a very significant paragraph on page 344, in which the author deals with the advisability of sacrificing, in some cases, the fractional-load efficiency of wheels for the sake of the economical advantage in initial cost of installation. This is of some interest, but the speaker thinks that the general principle laid down should be viewed with caution. As the statement stands, it is undeniably correct, as applied to developments such as those at Niagara Falls, where the maximum quantity of water that may be diverted is limited by law; and also where the quantity of water used at fractional loads is of no importance, being well below that which may be diverted.

It also applies well to those cases where the development is not greater than the minimum flow of a stream. In most installations, however, it will be found economical to over-develop the minimum flow, and to supply a steam reserve to take care of the periods of deficiency in the natural flow. In such cases the steam reserve will stand idle for the major part of the year. It becomes desirable, therefore, to minimize the fixed cost by installing a cheap steam plant (necessarily an inefficient steam plant). The wheels would operate at times under reduced flow, and, unless there is a multiplicity of units, enabling all the units in service at any time to be operated nearly at their full load, they will be worked uneconomically, because the plant is an uneconomical low-load plant. A considerable quantity of coal will be burned, under these conditions. Therefore, in a plant where the minimum flow is less than that required to carry the minimum load which may be thrown on the plant, it becomes desirable to look to the fractional-load efficiencies of the wheels.

Another comment by Mr. Larner, to which the speaker would like to call attention, refers to tests. The data contributed by the author are of so much interest and value that it would be a pity to overlook the facility with which tests on units connected to electrical generators may be made at various speeds, especially of wheels which are not amenable to such tests as may be made at the Holyoke flume.

There is not much mystery connected with the losses in electrical generators. Losses in the power going to waste in the conductors of the armatures of the machines are readily calculated by the application of Ohm's law to readings taken during the operation. The friction and windage in the machines are a purely mechanical proposition, and can be determined by tests in the field. The only other losses are those in the iron in the generators. These losses are a very simple speed

function, and can also be readily determined. As the total losses will not be more than from 2 to 4%, even in the larger generators, a slight inaccuracy in determining their variations with the speed is not very serious. Mr. Parker.

The readiest method known to the speaker for determining these losses as a speed function is by using what is known as a deceleration curve. With the water shut off from the turbine, the machine is brought up to speed by some outside source—possibly electrical—by interconnecting the generators on two wheels, one of which can take water; bringing them up to more than their running speed, and then cutting loose that wheel in which the electrical generator losses are to be determined.

It is then possible to determine losses by the electrical meters and by observing the speed of the wheel as a time function. Knowing the moment of inertia of the rotating parts—which can be readily calculated—observing the negative time differential of the speed, and multiplying by the moment of inertia, one can readily plot the power losses as a time function.

The moment of inertia can be determined experimentally in one of two ways. In a small machine, the addition of a known and readily determinable weight gives a ready solution. Its influence upon the deceleration curve will give the moment of inertia of the masses of the rotating parts under normal conditions.

Another method which the speaker has intended to try (but has not yet succeeded in doing because of lack of time) consists in using an electrical braking device. The simple connection of the electrical generator through a known resistance, and the measurement of the current carried by it, will show what the power usefully generated does in the way of braking the machine. Determining this moment of inertia and taking the whole group of curves should not take more than half a day, and therefore it would be easy to obtain the electrical losses as a speed function, and thereby extend to large units the very valuable study that Mr. Larner has made with such units as are capable of being tested in the Holyoke flume. The speaker believes that engineers who have facilities should look into the matter and see if Mr. Larner's data cannot be extended to such wheels.

Another of Mr. Larner's facts is of great interest, and the speaker is very glad that he has put himself on record as being opposed to the radical application of "standardization," a shibboleth which has been one of the most baneful things in practice, namely, that of taking "goods off the shelf"—the selling of stock goods for any and every purpose.

In building up American enterprises, standardization has been very helpful; but the speaker thinks that those who have had anything to do with the purchase of apparatus and who have attempted its intelligent

Mr. Parker. application, have met, in all lines of manufacturing, with the unfortunate use of "standards" to prevent progress.

The attitude taken by Mr. Larner—who is connected with the manufacturing industries—is certainly most hopeful, and gives promise of some advance in American practice toward standards which heretofore have prevailed almost exclusively in Europe.

Mr. Allen. C. M. ALLEN, ASSOC. M. AM. SOC. C. E. (by letter).—This paper is of great interest to the writer, because he is especially concerned in the behavior of water-wheels, not only at the Holyoke testing flume, but particularly after installation. The hydraulic turbine is by far the most efficient of any of the prime movers, and when one considers that to-day it is possible to obtain an efficiency of 90%, as shown by these tests, it certainly seems fitting that the turbine, including the setting, should receive as much intelligent and scientific attention as the steam engine, the steam turbine, or the gas engine.

During the past ten years, the writer has tested water-wheels after installation, for both horse-power and efficiency, using an Alden absorption dynamometer to weigh the load accurately, measuring the water sometimes with a standard weir and sometimes with a current meter. Tests have been made on wheels varying in horse-power from 50 to more than 4 000, acting under heads ranging from 10 to 200 ft., and running at speeds of from 75 to 800 rev. per min., and the writer is fully convinced that turbines can show as good an efficiency after installation as it was possible to show at the Holyoke testing flume, and that, in some cases, a better efficiency may be obtained. This is especially true in the behavior of high-head turbines; one runner, tested at the Holyoke flume, showed an efficiency of 83 or 84% at its best gate-opening and speed, and when it was tested after installation, under a 200-ft. head, it showed an efficiency of more than 86% in several instances. The load was measured very carefully with an Alden dynamometer, and the water was measured over a standard weir. A great many turbines composed of a pair of wheels have also been tested under heads varying from 20 to 50 ft., where the actual horse-power developed after installation was practically what the Holyoke tests would show when corrected for new head and speed conditions, according to the rule that, if the velocity ratio remains constant, the horse-power varies with the third power of the square root of the head.

There are also many wheels which do not come up to the rated power, as calculated from the Holyoke tests. In order to have wheels give their proper rating "in the field," it is necessary to give them good settings and to run them at the proper speeds. A good setting is a most important item, and means: That the water should be brought to the wheel with a low velocity; the draft-tube should be air-tight, and should be designed to take the water away from the individual runners with as little conflict as possible; in order to avoid obstruction

around the center of the buckets, the shaft should not be larger than necessary; the wheels—if a pair, and center-discharging—should not be set so closely together that the discharge of one interferes with that of the other; the bearings should be kept in line, etc. Mr. Allen.

Brake testing of turbines after installation is as essential to the designer of turbine settings, as the testing of runners at the Holyoke flume is to the designer of turbine runners. Many a good wheel has been cramped and mistreated in its setting, and, therefore, will never give its legitimate power and efficiency.

A great deal of information is still needed concerning the behavior of turbines installed in various settings, and under medium and high heads. If the efficiency of the plant as a whole is to be put on as high a plane as possible, more engineering effort should be exerted in securing efficient settings, and at the present time there is more chance in this way to increase the efficiency of the whole plant several per cent. than in any other.

E. KUICHLING, M. AM. SOC. C. E.—The author has presented in excellent manner a number of interesting features on the construction and performance of hydraulic turbines, most of which are inadequately discussed in textbooks and rarely found in manufacturers' publications. Occasionally, the report of some favorable test of a particular style of wheel is encountered, but the data are usually insufficient to enable one to determine satisfactorily the efficiency of the motor under varying conditions of speed and discharge. In this paper, however, Mr. Larner submits all data pertaining to a wide range of tests, made at Holyoke, of five turbines designed by him for the Wellman-Seaver-Morgan Company; he then shows how to compare properly the results attained for the several wheels, and, in conclusion, gives numerous valuable suggestions relating to the choice of turbines for different heads. Mr. Kuichling.

This contribution to the literature of American water-wheels is very welcome, even though it omits a consideration of the details of design on which satisfactory performance must depend. The author takes the ground that a manufacturer is justified in withholding from the public a thorough knowledge of the points of superiority in design, which he may be able to devise; but, in opposition thereto, it may be urged with much force that such a course tends to foster doubt as to the real merit of the invention, and to retard its general use. To the speaker it seems entirely reasonable that a purchaser should have a correct knowledge of the physical and mathematical principles on which a successful invention is based, in order that he may be able to select the best design or type for his purpose; and it usually happens that if such information is deliberately concealed, a more or less adequate empirical criterion will be developed, in course of time, by which a choice can readily be made. In general, the manufacturers

Mr. Kuichling.

who display the most thorough knowledge of the principles of mechanical design obtain the largest share of business.

Many features must be taken into consideration, in finally selecting a turbine for a particular locality and service, but only a few are involved at the outset. The quantity of water and the head available at different seasons of the year are known, approximately, as well as the desirable magnitude of the power units, but when it comes to determining the most efficient size, discharge, and speed of the turbines, much variation in practice is found. This, without doubt, is due to the lack of exact knowledge of the relations between the diameter, depth, discharge, speed, and power of American wheels, in which the buckets are usually of such complex form as to render a mathematical analysis of the motion of the water through them impracticable. It is necessary, therefore, to depend in large degree on the unpublished experimental results acquired by the wheel maker, and to exact of him a guaranty that his turbine will perform the required work at the stipulated efficiency. In the speaker's opinion, this condition is unsound, and should be replaced by a more extensive distribution of experimental data and the formulation of reliable principles of design.

The author has referred to his experiments for determining the velocity of the water at the top and bottom of the guides and in the draft-tube. Much difference of opinion as to the motion of the water in these places has been expressed by writers, and Mr. Larner, therefore, could increase the value of his paper greatly by adding some of his observations in these respects, such as the manner in which the velocity is distributed in a draft-tube of large diameter when the wheel is running at partial gate and full gate. It has been claimed by some engineers that a draft-tube cannot be made too large, while others advise the restriction of its diameter to a velocity of 10 ft. per sec. at full gate. Similarly, with the guides to the turbine buckets, some claim that these adjuncts are wholly unnecessary, while others attach great importance thereto. Experimental data in these directions are rarely encountered, and their presentation will doubtless be highly appreciated by all who are interested in hydraulics.

There are other obscure matters which Mr. Larner can doubtless elucidate, for example: Should there be any difference between the efficiencies of a right-hand and a left-hand turbine of the same type and size, under equal head and discharge? Is the power and efficiency of a turbine the same when it is set with its axis horizontal as it is when set with its axis vertical, it being assumed in each case that the surface of the head-water is far enough above the wheel to prevent any entrainment of air, that the head and discharges are equal, and that the wheel is similarly mounted, either in a casing or in a capacious open flume, and with like bearings and draft-tube?

DANIEL W. MEAD, M. AM. SOC. C. E. (by letter).—This paper is a valuable contribution to the literature of the hydraulic turbine, and deserves the careful consideration of every engineer interested in the development of water-powers. Mr. Mead.

As the author states, there is still room for vast improvement in the design and manufacture of the turbine. There may be little chance for securing greater efficiencies than are shown by some of the tests recorded in the paper, but such results are seldom obtained in practice under the actual operating conditions of speed and load. In other words, a well-designed and well-constructed turbine under the best test conditions will frequently give high efficiencies; but it is much more difficult to design a wheel so that such efficiencies will be secured under the fixed conditions of speed and load in the actual installation.

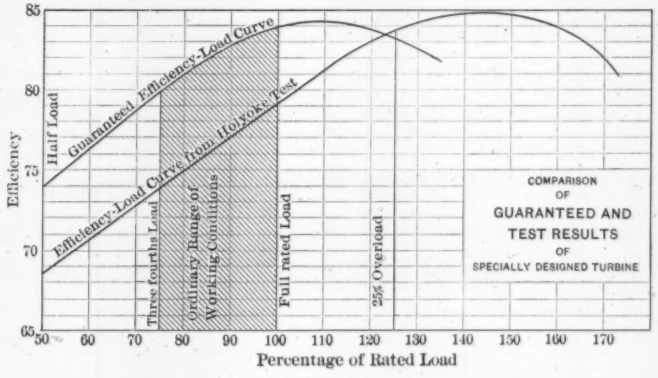


FIG. 20.

From the writer's observations, it is safe to say that the design and manufacture of water-wheels has not yet reached a stage where, without considerable experimental work, a wheel can be designed and constructed which will unquestionably secure the desired results under actual operating conditions. The only apparent exception to this is secured by the adaptation of wheels already experimentally developed, or "standardized," to conditions to the necessities of which their characteristics closely approximate.

This difficulty is well illustrated in Fig. 20, in which the guaranties for a recent installation are compared with the actual test results secured by the manufacturer with the first wheel especially designed and constructed for the fixed conditions. The maximum efficiencies secured on the test were above the efficiencies guaranteed, but the power of the wheel was too great, and therefore the efficiencies under the conditions of operation were deficient.

Mr. Mead. In the writer's opinion, the development of numerous experimentally perfected designs, or what are termed by many engineers "standard wheels," or "stock wheels," is of decided advantage, for it is found that by care in manufacture a successful wheel may be duplicated, enlarged, or reduced in size, without any great changes in its characteristics. With such wheels already developed, it remains for the water-power engineer only to see that the wheel selected is of proper size and character for the conditions under which it is to operate, and the results that can be attained are reasonably dependable.

The results attained in water-power installations are seldom better than the demand, and the carelessness or ignorance of the engineers in selecting turbines is as much a fault in this matter as the poor design or construction of the manufacturer.

In the past, too much dependence has been placed on the manufacturer and in guaranties made by him which are seldom verified by actual determination of results. The test of a plant under operating conditions is commonly exceedingly expensive, and often so difficult as to make it inexpedient. Knowing this, some manufacturers have frequently made unwarranted guaranties which could not be carried out—depending on the ignorance of the purchaser or his engineer, and the unlikelihood of a final test, to free them from the disastrous results of an unfulfilled contract.

It is perfectly true that the results of a test at Holyoke are only applicable to the same or a similar wheel under corresponding conditions of installation. When the wheel is equally well constructed, and equally well installed, the results that can be attained will approximate closely to the test results; and when the conditions of installation are different, the test results afford the best basis on which to estimate the results which may reasonably be anticipated.

Good mechanical work in water-wheel construction can be secured by close inspection, as in the case of any other mechanical work. The engineer, then, must select the proper wheel to fulfill the conditions of his installation and see that the mechanical details are properly executed.

In another place (see the author's reference at the foot of page 323), the writer has pointed out certain characteristic coefficients of water-wheels, and has discussed their use in the preliminary consideration of the selection of a water-wheel for a given installation. The writer's K_5 in that discussion corresponds to the square of the author's N_u, and is used in that way in order to facilitate calculation. In this discussion, however, the author's nomenclature is adopted so as to avoid confusion.

While, in a general way, the value of the "unit speed" of various wheels will give a ready means for their approximate comparison, it

must be remembered that all wheels having the same value of N_u are not necessarily of similar value in a given installation. Mr. Mead.

N_u is made up of two factors, namely, the "unit speed" under the "unit head" (1 ft.) $= \dfrac{N}{\sqrt{h}} = N'$ and the square root of the "unit power" under the "unit head," $\sqrt{P'} = \sqrt{\dfrac{P}{h^{\frac{3}{2}}}}$,

that is, $N_u = N' \sqrt{P'} = \dfrac{N}{h^{\frac{1}{2}}} \sqrt{\dfrac{P}{h^{\frac{3}{2}}}}$.

The value of N' enters directly into this value, while P' enters only as its square root. The value of N', therefore, has much the greater influence on the character of the wheel.

In a recent installation it was desired to develop about 650 h.p. with a unit of two turbines under a 16-ft. head at 128 rev. per min. Therefore, it was required that the following relations should obtain approximately for each turbine:

$$h = 16,$$
$$N = 128,$$
$$N' = 32.1,$$
$$P = \frac{650}{2} = 325,$$
$$P' = 5.07,$$
$$N_u = N' \sqrt{P'} = 32.5 \sqrt{5.07} = 72.4.$$

The turbines on which bids were submitted had the following characteristics:

Required $N_u = 32.5 \sqrt{5.07} = 72.4$,
No. 1... $N_u = 32 \sqrt{4.85} = 70$,
No. 2... $N_u = 30.2 \sqrt{5.2} = 70$,
No. 3... $N_u = 37.5 \sqrt{5.02} = 85$,
No. 4... $N_u = 35.7 \sqrt{4} = 71.4$,
No. 5... $N_u = 32.1 \sqrt{5.2} = 74.7$.

All these turbines, with the exception of No. 3, closely approximated the required value of N_u. No. 4, which most closely approximated the true value, had too high a value of N' and too low a value of P', and, therefore, was not suited to the conditions. No. 5 had individual characteristics which rendered it unsuitable. Nos. 1 and 2 most closely approximated the desired results, but No. 1, while slightly deficient in power, was highly efficient, and was selected as the most desirable wheel offered. (See Figs. 26, 27, and 28.)

The comments made above, however, could be based only in the most general way on the values of N_u, for it must be understood that the

Mr. Mead. values of N_u, or other turbine coefficients (as discussed in the previous reference), represent only the best results under one condition, that is, the point of highest efficiency at full or seven-eighths gate, as the case may be.

The engineer, while interested in the point of maximum efficiency, usually has to consider various other conditions of power (and hence of gate), and often variations in head must be taken into account as well. The engineer, therefore, must make a much broader analysis of operating conditions. For this purpose, a graphical analysis of an actual test has been found by the writer to be apparently the only method at once accurate and rapid by which the required information can be obtained. Two methods for such analysis have been pointed out by the writer in the discussion already referred to. One of these methods, and perhaps the simplest form, is that used by the author in this paper. Using either of these methods, the engineer can readily determine the results which he can expect to secure from a wheel in which the design is homologous to one for which test results are available. The results of actual analysis will perhaps best serve to show the desirability of such analysis and its necessity, if the best results are to be attained.

The tendency of manufacturers is always to furnish wheels of more than ample power, usually at a sacrifice of efficiency (see Fig. 20). This tendency is encouraged by the fact that capacity is readily determined, while the determination of efficiency is more difficult and frequently impossible. To guard against this tendency, analysis is necessary, and the specifications under which bids are invited should be clear and specific. In a contract let some time ago, the writer prepared, among others, the following specifications:

"Each unit shall consist of a pair of turbines in tandem, and shall be capable of developing a maximum of 1 900 actual horse-power under a working head of 70 ft. when running at 375 rev. per min. These turbines are to be designed to operate satisfactorily under a maximum head of 80 ft., and a minimum head of 65 ft. if so required.

"The contractor shall furnish a Holyoke test sheet of a turbine of homogeneous design to that he proposes to furnish. Said test should be of a turbine of similar size to the turbines on which proposals are made, but, if such is not available, it may be on the nearest size available."

On these specifications seven bids were submitted, and the guaranties shown in Fig. 21 were made. Two of the bidders failed to submit test data, and their bids were not considered. An analysis of the test data (Fig. 22) showed that the results which could be actually attained by the wheels offered were, according to the tests, in every case less than the guaranty. The wheels were in all cases too large for the best results, as is clearly apparent from the diagram (except in case of

wheel No. 1), and the points of maximum efficiency were, in most cases, far beyond the ordinary range of operation. Therefore, all bids were rejected, and the matter was taken up directly with the manufacturers of wheel No. 1 which came nearest to suiting the conditions of operation. A graphical analysis of test results of wheel No. 1 as originally submitted is shown in Fig. 23, from which it will be seen that this wheel was too large to give satisfactory results under the operating condition and was not of a satisfactory type. It was apparent that a wheel of less diameter and of a slightly different type would fill, Mr. Mead.

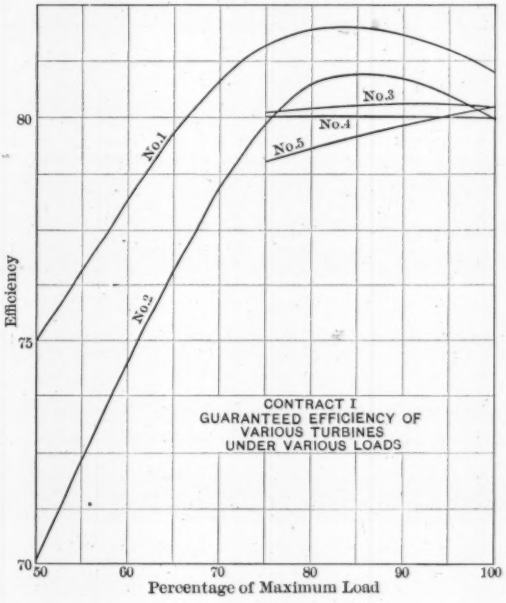


FIG. 21.

much more nearly, the conditions of operation, and a contract was entered into whereby the efficiencies shown in Fig. 24 were guaranteed. The results were to be demonstrated by a Holyoke test and by a test in place, if such was desired. The Holyoke test was to be made at the expense of the manufacturer, in the presence of the engineer of the purchaser; and the test in place, if made, was to be made at the expense of the purchaser.

It may be remarked that final surveys, completed before the contract was let, developed the fact that a maximum head of 85 ft. could

Mr. Mead.

be obtained. It also became apparent that, to secure the best results at all heads, less power than specified at the 70-ft. head was desirable.

With the wheels selected, four units at full load and maximum head will give the designed capacity of the plant, and five units (which includes a reserve unit) will give the full capacity of the plant under the minimum head, which, however, seldom obtains.

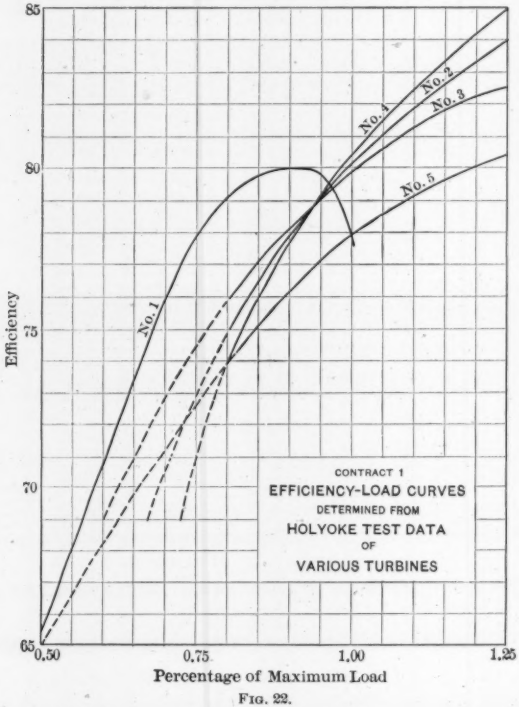


FIG. 22.

It was evident from the bids received that the specifications were not sufficiently explicit, and, while the desired results were secured before the final award, the specifications were clearly faulty.

In a more recent installation, an attempt was made to remedy the defect in the foregoing specifications, and the clauses referring to conditions of operation were prepared as follows:

"Bids are desired on three units of two turbines each. Each unit shall consist of one pair of turbines mounted on a horizontal shaft. These turbines will be installed in open penstocks, and shall be capable of developing a maximum of about 650 actual horse-power under a

Mr. Mead.

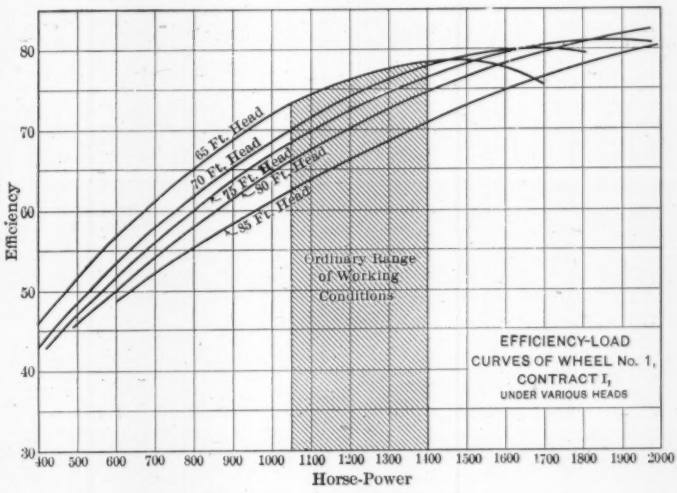


FIG. 23.

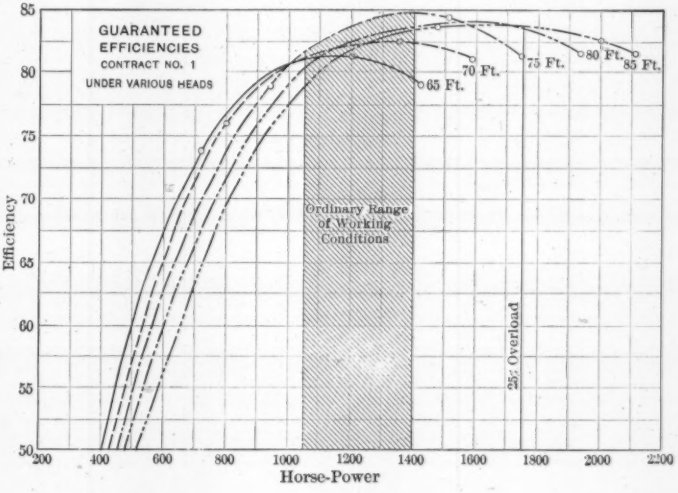


FIG. 24.

Mr. Mead. working head of 16 ft., and running at approximately 128 rev. per min. These turbines are to be designed to operate satisfactorily under a maximum head of 17 ft. and a minimum of 12 ft., if so required. The efficiency of the units at each condition of load shall be not less than that guaranteed by the bidder in his proposal.

"Each turbine unit is to operate a 60-cycle, 3-phase alternator of 360 kw. rated capacity at 90% power factor and 94% generator efficiency, and to a maximum of 25% over-load.

"The generator will be operated at from 75% to full rated load for the larger portion of the time, and only occasionally under over-load conditions. While the highest practicable efficiency is desired under all conditions of load, it is especially desirable to secure the highest efficiency under the most usual and continued conditions of operation. It is therefore desired that the size and type of wheels be so selected that the best practicable efficiency will be obtained between 390 and 520 h.p. of the turbine load (see Fig. 25).

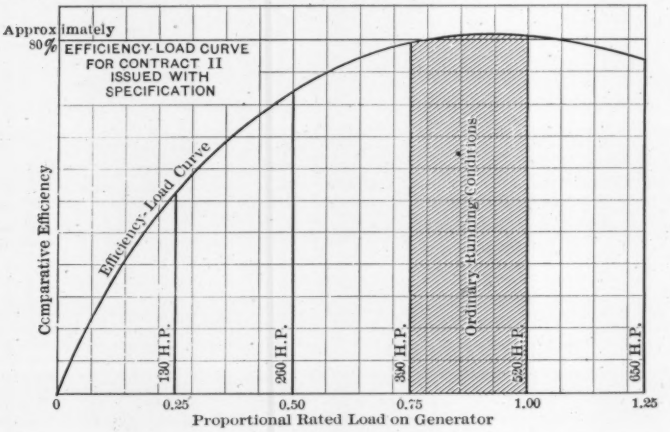


FIG. 25.

"The bidder must furnish with his bid a Holyoke test sheet of a turbine of homogeneous design and of the same size that he proposes to furnish. If a test of a turbine of the same size as that proposed is not available, then the bidder may furnish two test sheets of the two sizes nearest that proposed, which are available.

"The efficiency of turbine units shall be determined in the usual manner by a test at the testing flume of the Holyoke Water Power Company, in the presence of the Engineer, or his representatives. One main turbine unit furnished under this contract will be selected by the Engineer, and shall be carefully tested to his satisfaction, and the efficiency shown by these wheels will be the basis of calculating the efficiency of the unit.

"These tests shall be made at the expense of the contractor.

"If, after the Holyoke test is made and found satisfactory, and, after the turbines are erected, the turbine units fail to develop the power specified, or are found to use too much water for the amount of power generated, then, and in that event, any or all of the turbine units may be tested in place. The test shall be made by the Chief Engineer of the ——— Power Company, or by his authorized representative, and the turbine manufacturers shall be notified and shall have the opportunity of being present and taking part in said test. Mr. Mead.

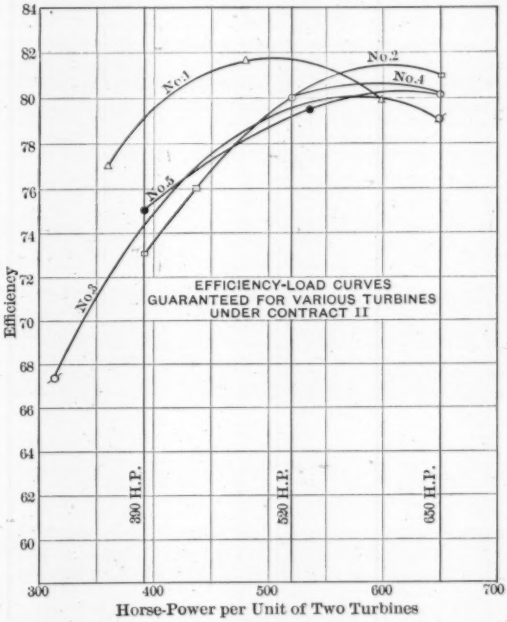


FIG. 26.

"The methods of making such tests shall be by means of recognized standard methods of measurement of water, power, and speed, and will be agreed upon, as far as practicable, by the parties hereto. If any disagreement arises, the decision of the Chief Engineer shall be final. The test in place will be made at the cost and expense of the ——— Power Company. Any turbines found defective by the test at Holyoke, or by the test in place, shall warrant the rejection of such turbines."

The diagram accompanying these specifications is shown in Fig. 25. The guaranties submitted with the bids are shown in Fig. 26, and the analytical results of the test sheets are shown in Fig. 27. It will be noted from Figs. 26 and 27 that in two cases the guaranties were

Mr. Mead. higher than shown as possible by the test results. The contract was awarded to the manufacturer of wheel No. 1, as the lowest and best bid. A comparison of the guaranty and test results of this manufacturer is given in Fig. 28, which shows that the guaranty is conservative and will probably be exceeded if the mechanical workmanship of the wheel is carefully executed. It may be noted that the above wheels are the same as those used in the discussion of the value of N_u.

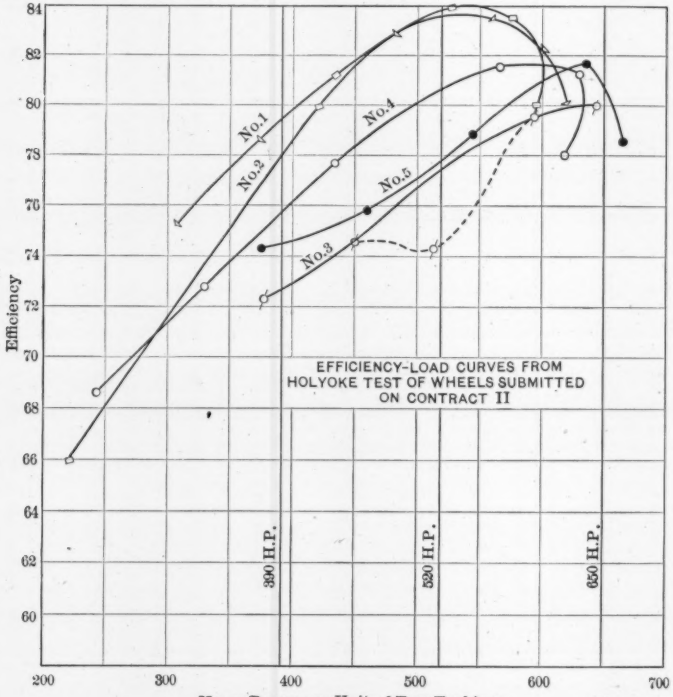


FIG. 27.

The latter form of specification is believed to be in general satisfactory. The Holyoke test specified is somewhat expensive, and its cost will usually be added by the manufacturer to the price bid. The same results may often be attained, without the actual expense of a test, by requiring a test of the completed wheel at a fixed price, provided the engineer so elects. This will assure care in manufacture, and the expense may be eliminated by the engineer if on inspection

of the completed wheel he believes its mechanical construction is equal to that of similar wheels that have already been tested. Mr. Mead.

Close analysis and rigid and intelligent inspection will assure a higher grade of design and workmanship than is commonly offered for water-power installations. The engineer who neglects these precautions and blindly follows the recommendations and unverified guaranties of manufacturers must expect to secure such unsatisfactory results as the contingencies of close competition and the temptation of the possession of standards already developed (but probably not strictly applicable to the condition) always create.

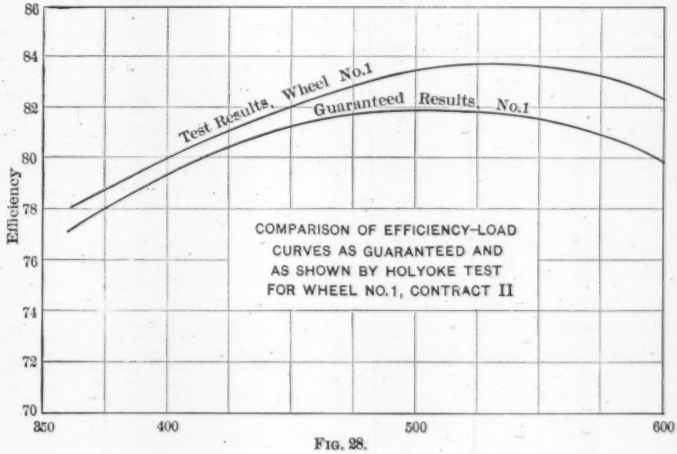


FIG. 28.

In many low-head plants the variations in the head under various conditions of river flow are often considerable. During low-water conditions the head may be a maximum, but with high water, on account of flowage conditions, the head-water must commonly be kept down by the use of a wide spillway section or flood-gates, while the tail-water will rise and hence the head will be reduced. When (without auxiliary power) it is necessary to run the plant under all these conditions, the detailed analysis based on test results of the wheels offered is the only basis from which their probable power and efficiency under the extreme range of conditions can be determined. Take, for example, the author's diagram, Fig. 12, of the test of a 31-in. wheel, and consider its use under heads varying from 16 to 9 ft. The relative unit speeds under the maximum and minimum head will vary as $\sqrt{h}$, and will be as 4 is to 3. If the wheel is arranged to run at 162 rev. per min., the values of N' will be 40.5 for a 16-ft. head and 54 for a 9-ft. head, which, at both maximum and minimum heads, will result in less than

Mr. Mead.

the best efficiencies. The actual efficiencies and power under the full range of conditions and for all intervening heads, can readily be calculated from the graphical diagram.

It is to be noted that as high efficiency (and consequently economy of water) is more essential at low water (and high-head conditions), a better speed for the above wheel might be 176 rev. per min., under which conditions the value of N' would be:

$$N' = 44 \quad \text{for a 16-ft. head,}$$
$$N' = 58\tfrac{2}{3} \quad \text{for a 9-ft. head.}$$

While this change would increase the power and efficiency slightly at a 16-ft. head, it would reduce the power and efficiency considerably at a 9-ft. head.

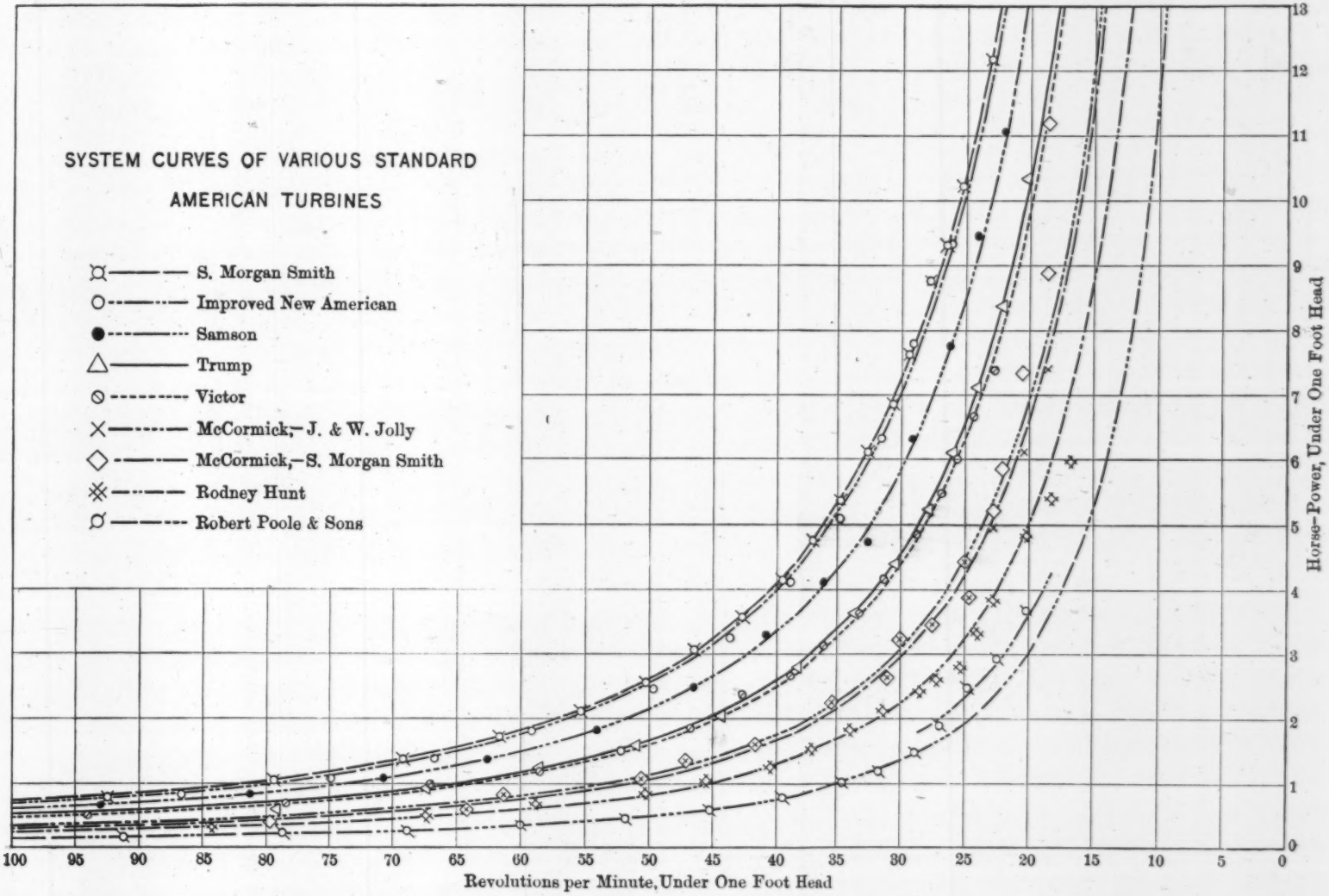
The foregoing comments will illustrate one of the points to which the writer wishes to call attention, that is, that the "specific speed" or "unity speed" of a wheel at the point of maximum efficiency, while of value in comparing test results and sometimes in the preliminary consideration of the general requirements of the conditions of operation, is of comparatively little value in the selection of a wheel for the varying conditions of operation.

Another point, on which the writer is inclined to take issue with the author, is on the "standardization" of wheels. In the present state of the art, the writer believes that the design of special wheels is too uncertain and their development too slow and too expensive to be warranted, except under unusual conditions. The development of standard construction means at its best the development of a line or type of wheels from which definite results may be realized quickly at limited expense. Every manufacturer must develop, by careful design and expensive experiments, lines or types of wheels which become valuable for broad application only as they are sufficiently standardized to assure the prompt fulfillment of definite results. Standard wheels already on the market differ widely in their characteristics. No one type of wheel is suited to every condition, but the wide variation offered by the several makers affords a range from which the engineer may readily select one or more wheels that will often fulfill in a satisfactory manner the conditions of his installation. The range of the standard wheels of various manufacturers is shown on Plate XXXIII. This diagram is computed from the catalogue recommendations of the manufacturers, which are approximately, but not always exactly, correct.

The writer believes that standardization is objectionable only when attempts are made to force the conditions to conform to the standards, instead of selecting or altering the standards to conform to the conditions.

Satisfactory results can be secured only by the ability of the purchaser's engineer to analyze the situation and the basic tests on which any intelligent bid of manufacturers must be based.

PLATE XXXIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXVI, No. 1138.
MEAD ON
MODERN HYDRAULIC TURBINES.



Mr. Zowski.

S. J. Zowski,* Esq. (by letter).—Mr. Larner and the Wellman-Seaver-Morgan Company deserve much credit for having abandoned the policy of the majority of the American turbine builders in regard to publishing the results of tests of newly developed turbines. That the policy of reticence was not a wise one was pointed out frequently to the interested parties, but with little success. Let us hope that the good example of the Wellman-Seaver-Morgan Company will have more effect and will be followed soon by others.

The results obtained by the Wellman-Seaver-Morgan Company's turbines, Holyoke tests Nos. 1795 to 1800, are remarkable: 90% best efficiency for medium-speed, medium-capacity turbines, and 86% for high-speed, high-capacity turbines, are record-breaking values, and both the designer and builder are to be congratulated.

It is a pity, though, that the author did not give any information about the wheels themselves. Many hydraulicians interested in the theory of the water turbine would like to utilize these experiments for a further advancement of the science of this art. They would like to make some computations and compare the results of these experiments with the theoretical formulas. It is quite natural that they would like to discover whether the good results were obtained by an improved runner design, or whether they must be attributed to the good performance of the draft-tube or guide-case, or to the good workmanship, or finally to the good mechanical efficiency of the machine. To make such investigations possible, the author should have given at least the principal turbine dimensions, such as the height of the guide-case, the upper and lower draft-tube diameters, the number of vanes and buckets, and the thickness of their tips, the actual (not only proportional) gate openings, and the bucket angles. Also, full information should have been given about the workmanship of the runner and case. It is true that the manufacturer could not reasonably be expected to give such details of design on which the obtained good results depend, but no broad-minded engineer will consider the above-mentioned main dimensions as responsible for a given failure or success.

The author mentions that the flow conditions in the draft-tube have been observed very closely, but he does not give any information on that important point; and yet it would be especially interesting to know at which gate opening and speed the flow in the draft-tube was not whirling, thus obtaining the so-called "perpendicular discharge," and whether there was any speed and gate opening at all at which this was the case over the entire draft-tube area. This information would show at once whether the high peak in the efficiency curve is due to the fact that the best speed diagrams at the entrance (shockless entrance) were obtained at the same time at which the

*Professor, University of Michigan.

Mr. Zowski.

best speed diagrams at the discharge (perpendicular discharge) were prevailing. It would also show whether any changes in these speed diagrams would raise the peak still more, or whether they would flatten the curve for the benefit of other gate openings, and it would indicate in which direction changes in the bucket angles and curvature should be made for a desired effect.

It is thought that in all publications of turbine tests which are made not solely for commercial reasons, the following rule should be adopted: That, in addition to the test sheet there should be given a sketch of the turbine with all main dimensions, and that the nature of the flow at the different gate openings and speeds should be indicated clearly.

All European turbine tests, for instance, those of Professor E. Reichel, of Charlottenburg, and in general all other experiments of scientific value, are reported in such a way. Why should an exception be made with American water-turbine tests? Undoubtedly, the observation of this rule would help to eliminate the controversies which exist between American and European turbine designers, and it would also help to eliminate all mutual accusations as to the reliability of the reports on the one side, and lack of knowledge and success on the other.

In Fig. 1, representing the high-speed turbine, it will be noticed that the water flowing along the lower gate ring must make quite a sudden turn in entering the runner. This, in connection with the high efficiency obtained in spite of the turn, would almost suggest that the important rule of hydraulics of avoiding all sharp turns does not apply to water turbines, which, of course, is not true—or else the turbine as a whole is so perfect that the loss undoubtedly caused by the sharp turn is balanced by other gains.

In speaking of the classification of water turbines as to speed, Mr. Larner makes the statement that in Europe the classification is done on the basis of peripheral speed. This statement is not quite correct, as it was in Europe that the method of classification on the basis of N_s (the author's N_u) was introduced first by Professor R. Camerer, of Munich, and Mr. Baashuus, and where, since 1905, it has been in general use. In the same year it was brought to the United States by European turbine designers who have transplanted their activities to America. Before that time, another characteristic, equivalent to N_u, but based on the discharge instead of the power, was used in Europe, also the "specific diameter" for which the author uses the symbol D_u, has been used by European engineers for the last five years. The first point considered by European engineers, in connection with turbine runners, is always the value of N_s (N_u). In the second place, comes the question of the peripheral speed, the coefficient of

Mr. Zowski.

which, ϕ, is a very convenient runner characteristic, indicating at once whether a certain high or low value of N_u is due more to a high or low speed (large or small bucket angles), or to a high or low capacity.

Although the boundary line between the different types cannot be drawn too sharply, and is subject to the fancy of the designer to a certain extent, the writer can scarcely agree with Mr. Larner in calling the runner of test No. 1795, a low-speed runner. Both the value of N_u and of ϕ make this runner belong to the medium-speed, medium-capacity type. Low-speed runners would have the following values, $N_u = 10$ to 28 and $\phi =$ about 0.58 to 0.65 (0.7). Therefore, Mr. Larner's statement that the most efficient speed varies with the gate opening, increasing as the latter increases, and that this is true of all reaction turbines, whether of low or high speed, is not justified by the tests, Nos. 1795 to 1800, as these tests covered only medium- and high-speed wheels. If the writer's memory is correct, experiments on low-speed turbines have not yet been made in this direction, or, at least, such experiments have not been published. That the best speed of medium- and high-speed runners increases with increase in gate openings is a well-known fact, established by theoretical analyses and many experimenters long ago.

In general, it might be said that there is a wide field for modifications and different solutions of the problem of variation of speed with varying gate openings. If, for instance, the main object is to obtain a flat efficiency curve, or, in other words, a so-called flexible runner, the runner should be designed in such a way as to obtain the best speed diagrams at the entrance (shockless entrance) at one, say, normal gate opening, and the best speed diagrams at the discharge (perpendicular discharge) at another gate opening. It is not necessary to state that this will reduce somewhat the absolute best efficiency.

In close relation to this is the question of the best efficiency gate opening. It is true, as stated by the author, that with low- and medium-speed turbines the best efficiency gate opening is usually at 0.7 to 0.75, whereas with high-speed turbines this is at 0.8 to 0.85. The reason for this difference is the characteristic of the gate-efficiency curves for the different types on the one hand, and the desire of getting the best possible performance at full gate, on the other hand. With the flatter curves of the first two types this last is easily obtained, even if the peak of the curve is at 0.7 gate, whereas with the more peaked curve of the latter type the drop in efficiency would be too large if the peak were at the same distance from the full gate as before. It should be understood, however, that it is in the hands of the designer to have also with high-speed turbines the best efficiency at 0.75 gate, if this is preferable.

Mr. Larner. CHESTER W. LARNER, ESQ. (by letter).—Professor Moody's chart, Plate XXXII, comprising the best results obtained by the representative builders of America and Europe, is a very valuable contribution to this subject. The writer is particularly glad that it was submitted in connection with this paper, because he desired to prepare such a chart himself at the time of writing the paper, but realized the futility of attempting to secure the necessary information from his competitors.

This chart fully substantiates the statement, made in the paper, that the results published there are the best thus far obtained in the practice of the art. The efficiencies shown by the five tests presented are higher in each case than the efficiency of any other wheels of corresponding capacity and speed. In fact, through the range of "medium" and "high-speed" wheels, the curve of maximum efficiency is determined by the writer's points. Furthermore, it should be noted that these points are from a consecutive series of tests covering a wide variation of design, and do not represent, as do all the other points plotted, the best results culled from a collection of relatively inferior ones.

The only points from the paper which lie below Professor Moody's curve are 1800 and 1795. The former (efficiency 88.6%), as previously explained, was an experimental wheel of incongruous design, and was not expected to show maximum results. The other point, 1795 (efficiency 87.7%), does not do full justice to this wheel, as compared with wheels of the same type in operation under high heads, for the reasons explained in the paper; and if it had been tested under a head as high as that under which the two tests, *A* and *B*, of the I. P. Morris Company were made, there is little doubt that this point also would lie on or near the curve. Professor Allen gives interesting evidence corroborative of this statement when he cites the case of a high-head wheel which he tested in the Holyoke flume before installation and afterward in place, and found that the efficiency in operation was several per cent. better than that shown in the testing flume. This evidence is interesting because it is only in exceptional cases that such a direct comparison can be made, but the conclusion arrived at is so clearly obvious from a consideration of the elementary principles of mechanics, that experimental proof is almost superfluous.

Points *A* and *B* of the I. P. Morris Company, while speaking very highly for Professor Moody's ability as a wheel designer, testify also to the truth of the writer's contention regarding the advantage of testing wheels under high heads, because the best result obtained by this company, out of all the tests it has made in the Holyoke flume, is point *C*, which is about 3% lower than its best test in place, namely, point *B*. The writer is gratified to note that Professor Allen, who has

had such a wide experience in testing water-wheels, agrees with him on this point. Mr. Larner.

Another interesting point brought out by this chart is the exceptionally high specific speed of the writer's wheel, 1796, as compared with the known records of other high-speed wheels. At a value of $N_u = 100$ it shows an efficiency of practically 80 per cent. The only other wheel which has ever attained as high a value of N_u is one designed by Professor Wagenbach of Charlottenburg.* The efficiency shown by this test was much lower than test 1796, as can be seen by reference to Professor Moody's chart, or to Fig. 29, where the efficiencies of both wheels are plotted for three different values of N_u.

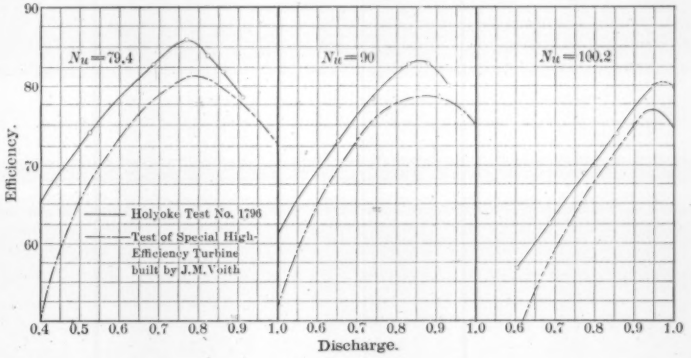


FIG. 29.

In order to show the real significance of the results obtained thus far in high-speed wheels, Table 10 has been prepared showing the maximum amount of power which could be produced at an efficiency of 80% by each of the four wheels on Professor Moody's chart which show the highest specific speeds. The power is computed for one runner at 150 rev. per min. under 50 ft. head. The values of N_u are taken from the chart.

TABLE 10.

Name of wheel.	N_u at 80% efficiency.	Brake horse-power.
Wellman-Seaver-Morgan Co., 1796	99	7 700
Wagenbach experimental	88	6 100
Allis-Chalmers Co., 1778	83.5	5 480
Swedish wheel (Curve *H-I*)	83	5 400

* *Engineering News*, September 9th, 1909.

Mr. Larner.

These values are obtained by substitution in Equation 2, which becomes:

$$\text{Horse-power} = \left(\frac{N_u \times 50^{\frac{5}{4}}}{150}\right)^2 = \left(\frac{N_u \times 133}{150}\right)^2$$

These figures will hold the same relation for any head or any speed, and, therefore, it may be stated in general that at any given speed and under any given head, wheel 1796 will develop about 40% more power at an efficiency of 80% than any other wheel of American manufacture, and about 25% more than the highest speed wheel thus far produced in Europe.

The writer is unable to appreciate the logic of Mr. Kuichling's objection to the statement made in the paper that the details of design and methods of obtaining results concern only the manufacturer who uses them. There are two chief reasons for this statement, each sufficient in itself to justify such an attitude. The first is that points of superior design in turbine runners require as much thought, originality, and outlay for experiments as the most complicated inventions in other lines of work. If the protection of a patent is sanctioned in the latter case, why should not a similar protection be justifiable in the former. Such improvements are not patentable, but as long as the builder keeps his designs to himself they are in a measure protected.

The second reason is the utter uselessness of attempting to educate the operating engineer to an intelligent choice of turbines through a study of variations in runner designs. It is the most complicated subject in mechanical engineering with which the writer is acquainted, and it would be a hopeless task for anyone but a specialist to acquire a sufficient knowledge of it to be of any practical value. Furthermore, the writer is fully convinced that most engineers do not care anything about such matters. They have neither the time nor the inclination to study them. They want to know as much as possible about the practical results which can be obtained, but they are satisfied to leave the methods of arriving at those results to the designer.

This, of course, is not true of all kinds of machinery, and the writer is inclined to think that Mr. Kuichling is partly right in what he says, but makes his mistake in that he does not distinguish between what is practicable and what is not. There is a great deal of machinery which embodies in its design only the simpler principles of mechanics, on which any competent engineer can readily pass judgment. Most of the mechanism of a turbine falls in this category, and no builder makes any secret of this part of his design; but the guides and runner are entirely different, and they are of more importance than any other parts. There is no living man who can tell with any degree of certainty, by merely looking at a runner, what the characteristics of its

performance will be. There is no definite law, as Mr. Kuichling suggests, governing the relations of the diameter, depth, discharge, etc. These relations are entirely different for wheels of different types, and even for wheels of the same type, that is, the same unit speed. All these proportions vary with the design of the wheel vane, and the latter is an intangible factor to the average engineer. In fact, if a drawing of a wheel vane were put in evidence, there is not one engineer in a thousand who would know what he was looking at. Mr. Larner.

As to other features of turbine design which can be more readily analyzed, the writer agrees with Mr. Kuichling that more data are desirable. One point raised by him is the proper size of the draft-tube. He states that some engineers are of the opinion that it cannot be made too large, while others believe that it should be of such a size as to give a velocity of 10 ft. per sec. at full gate. When two such contradictory and equally erroneous theories are abroad, it would certainly seem that more light is needed.

The design of a draft-tube is no very complicated matter. The controlling factor in the design should be a uniform reduction of velocity from the runner to tail-water, and, in speaking of the "draft-tube," the writer includes the entire channel from the runner to tail-water. Bearing in mind the principle of uniformly reduced velocity, the problem becomes one of determining the diameter of the upper end of the tube, the diameter of the lower end, and the length.

The diameter of the upper end should be the same as the diameter of the discharge end of the runner, as shown in Figs. 1 and 2, and if, as in Fig. 1, the wheel is of large capacity, with some outward discharge, it is well to give the tube a slight flare at this point, although this can be easily overdone. There should be no sudden enlargement from the runner to the draft-tube, because such an increase in cross-section tends to check the velocity of discharge suddenly and create a disturbance of the stream. This principle of design is frequently violated, both in America and Europe. The writer has seen on turbines discharge cases which looked more like dry-goods boxes than anything else, which did not even approach the circumference of the runner, and made no pretense whatever of providing for uniform reduction of velocity. The runner discharged into the draft-chest like a jet from a hose discharging into a pond, and with about the same efficiency. One installation of this description which the writer has in mind was finally consigned to the scrap pile because the efficiency was found to be only about 65 per cent. The runners showed an efficiency in the Holyoke testing flume of more than 80%, and this astonishing discrepancy was probably due almost entirely to this particular feature of the installation.

The diameter of the draft-tube at the lower end is dependent to a considerable extent on the diameter at the upper end, and the length

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of the tube. The latter is frequently fixed by conditions independent of the turbine, and as there is a limit to the amount of flare desirable in a draft-tube, the diameter of the lower end sometimes has to be made smaller than one would wish. The writer considers a flare of 1 ft. in diameter to 3 ft. in length about the limit, and prefers less than that if possible. The criterion for judging whether or not a draft-tube is too small at the discharge end is the percentage of energy lost in velocity at that point. Under low heads this loss can seldom be kept below 1%, whereas, under high heads, it is usually a fraction of 1%, but only because high-head wheels are usually of the reduced discharge type, and hence, on account of the relatively smaller quantities of water to be handled, the velocity of outflow from the draft-tube can be reduced to a relatively lower figure without making the tube of prohibitive proportions. For a given flare, the longer the tube the lower the velocity of discharge, and hence the more efficient the tube; remembering, however, that a safe limit for the elevation of the top of the runner above tail-water, when set in a horizontal position, is about 25 or 26 ft. In the case of low-head plants, where the wheels, in order to be sufficiently submerged, must be set close to tail-water, it is often necessary to form the draft-tubes in the foundations of the power-house, and to carry these passages sometimes below the level of the tail-race and come up to that level after the discharge from the wheels has been released to the atmosphere. It should be remembered that the effective draft-tube extends to the point where the water is released, and that the energy lost in residual velocity is the velocity head at that point. The percentage of energy lost is:

$$\frac{V^2}{2g} \div h \qquad (4)$$

where V is the residual velocity and h is the working head on the plant.

From these considerations, it must appear that no fixed figure can be named for the desirable velocity in a draft-tube. The velocity depends entirely on the head. Suppose, for example, a wheel is operating under 9 ft. head. It discharges 100 cu. ft. per sec. Assuming that the maximum size possible for the discharge end of the draft-tube, without excessive flare, is 5.7 ft. in diameter, then the residual velocity will be 2.4 ft. per sec., which, according to Equation 4, is:

$$\frac{2.4^2}{2g} \div 9 = 0.01, \text{ or } 1\% \text{ loss.}$$

Now suppose the head on the wheel to be increased to 900 ft. If the speed is allowed to change correspondingly, the discharge will vary as the square root of h. The discharge will become:

$$\frac{100 \times \sqrt{900}}{\sqrt{9}} = 1\,000 \text{ cu. ft. per sec.}$$

The residual velocity will then be $2.4 \times 10 = 24$ ft. per sec., and the loss, according to Equation 4: Mr. Larner.

$$\frac{24^2}{2g} \div 900 = 0.01, \text{ or } 1\% \text{ loss},$$

as in the previous instance. Hence it is plain that a certain size of draft-tube, in connection with a certain runner, will give the same draft-tube loss, no matter what head it is used for, and that the velocity of outflow can have no arbitrary or empirical value, but must vary with the head.

As to the relative efficiencies of right-hand and left-hand wheels, there certainly should not be, nor is there, so far as the writer knows, any difference. The records seem to show that there are about as many efficient wheels of one type as the other. There is no theoretical reason to believe otherwise, and the only alleged reason that the writer has ever heard is a fanciful notion about the effect of the earth's rotation.

There is also no reason to believe that there is any difference between the efficiencies of wheels set in horizontal or vertical positions, or even in an inverted position. If the head is the same in all positions, the discharge will be the same under like conditions of gate and speed, and if the discharge is the same, then all the hydraulic conditions are duplicated and the efficiency must be the same. This, of course, refers to the efficiency of the runner alone, because the efficiency of the turbine may be very different on account of the mechanical losses incidental to different settings. In this respect the advantage is usually in favor of the vertical setting, because the weight is ordinarily carried on practically frictionless bearings, which is not the case in horizontal settings.

The only test to determine this point, of which the writer knows, was made some years ago at the Holyoke flume on wheels for the Michigan-Lake Superior Power Company at Sault Ste. Marie, Mich., one of the oldest installations of the Wellman-Seaver-Morgan Company. These wheels were tested both ways, and as there were forty-four units of four runners each, a great many were tested. Some were tested singly on a vertical shaft and then in pairs on a horizontal shaft, in the casings in which they were to be permanently installed, and it was found that the efficiency was the same in each position, provided the wheels were placed far enough apart on the horizontal shaft. If placed too close together, the loss of efficiency was very marked. It is to be regretted that more tests of this sort have not been made, for the purpose of checking the tendency of many builders to crowd their wheels together in order to shorten the shaft and effect a corresponding "economy" in design.

Referring to Mr. Kuichling's request for data in regard to the

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distribution of velocities in the draft-tube and guide passages, as determined by Pitot tubes, the writer believes there is sufficient material in this subject for a separate paper, and, when the opportunity affords, he intends to prepare some of this information for publication. For the present it may suffice to say that the phenomena observed in the draft-tube are in general about as follows: Starting with the wheel held stationary and the gate open, the velocity of the water at the center of the tube is very high, and at the circumference it is low. The water is whirling at all points on the diameter in a direction opposite to the direction of rotation of the runner. Both these phenomena would naturally be expected. The incoming stream from the guides, having an initially radial direction, naturally crowds to the center and causes a high velocity there. When the wheel starts to revolve, and as it increases in speed, the centrifugal force of the revolving mass of water in the wheel tends to counteract the first condition. The water presses outward toward the band of the runner and the velocity across the diameter of the draft-tube becomes more evenly distributed. The distribution, however, is never uniform. The velocity in the draft-tube, as in any other pipe, is always highest at the center except at very high speeds, when the effect of the centrifugal force is so great that there is almost a void around the shaft.

The angle of whirl when the wheel is stationary is backward, because the flow of the water relative to the wheel is backward at all times, but is offset, when the wheel is running, by the velocity of the wheel in the opposite direction. When the velocity of the wheel at any point on its diameter is equal to the horizontal component of the velocity of discharge relative to the wheel, then the velocity of discharge relative to the draft-tube, or the absolute velocity, will be straight down, and there will be no whirl. As the speed of the wheel increases beyond this point the water begins to whirl with the wheel. This is just what is observed in testing with the Pitot tube.

One very interesting fact shown by these experiments is entirely at variance with the accepted beliefs regarding the action of the water in the runner. It is generally supposed that "radial outflow" or discharge without whirl occurs at the most efficient speed of the wheel. The idea is that because radial outflow is the theoretical condition for maximum efficiency, therefore the most efficient speed will be the one where radial outflow* occurs. The tests, however, show that this is not the case. The speed at which whirl is eliminated from all parts of the discharge area, except that close to the center of the shaft, is much lower than the most efficient speed. The water starts to whirl with the wheel at a comparatively low speed, and increases until it

* The term "radial outflow" is commonly used in reference to all types of wheels, but is strictly correct only in reference to purely inward- or outward-flow wheels. For mixed-flow wheels, the correct term would be "axial outflow" when applied to the stream after it has emerged from the band of the runner.

shows an angle of from 15 to 30° from the vertical. This value varies with different wheels, but remains fairly constant up to speeds much higher than the best speed. The explanation apparently is that centrifugal force, increasing with the speed, accelerates the backward velocity of discharge about as fast as the forward velocity of the wheel increases. Mr. Larner.

At the point of maximum efficiency, the conditions observed were about as follows: The velocity was highest at the center of the draft-tube and lowest at the circumference. The water had no whirl whatever at the center, as far as could be measured. At the circumference it was whirling with the wheel at an angle of from 15 to 30 degrees. These conditions shaded into each other at intermediate points. The water seemed to flow in a quiet, solid mass, without the conflicting angles of whirl and velocities which were observed at other speeds.

The writer is convinced that one of the chief losses of energy in a turbine occurs at discharge from the runner, and that most poor wheels owe their inefficiency more to an improper distribution of the discharge area between the vanes than to any other cause. If the discharge opening is not properly proportioned, the adjacent angles of whirl and velocities of discharge will conflict, causing great disturbance.

The effect of gate opening on the flow in the draft-tube is to increase the velocity at the center. This, to a certain extent, accounts for the fact that the most efficient speed of the wheel increases with the gate opening. Each time the gate is opened farther the water tends to crowd more to the center, and hence more centrifugal force, requiring a higher speed, is needed to secure the proper distribution of flow across the diameter of the draft-tube.

The writer notes that Mr. Mead, in his interesting discussion, takes exception, in some respects, to the use of unit speeds as a criterion of the fitness of a wheel to perform certain work. Mr. Mead says that "all wheels having the same value of N_u are not necessarily of similar value in a given installation."

The writer did not claim that all wheels having the same unit speed are equally desirable, but that all such wheels are able to perform the same work. In other words, if an engineer intends to buy some turbines to operate under conditions of head, speed, and power which require a unit speed of 60 at full gate, and several builders submit tests of runners which show a unit speed of 60 at full gate, then he knows with absolute certainty that all these tests represent types which will produce the power he wants at the specified speed and head. The relative desirability of the wheels depends on their respective efficiencies.

The required wheels may not be of the same size as the wheels that

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were tested, but there can be no shadow of doubt that some size of each of these types will do the desired work, and it is a simple matter to determine the sizes. Mr. Mead admits that changes of size can be effected with greater certainty than changes of design, and therefore it seems to the writer of far greater importance to determine whether the builder has developed a wheel of the right type, even though it is of considerably different diameter, than it is to determine whether he has a wheel of the desired size but of only approximately correct type. It was for this reason that the writer tried to emphasize the point that unit speed is an index of type and should be so regarded to be of the greatest practical use. In the first case, just referred to, the builder needs only to alter the size of his wheel, whereas in the second case the other builder would have to alter his design, and the outcome would be far more uncertain.

Mr. Mead illustrates his use of N_u in the selection of turbines by reference to an installation under the following conditions:

Head = 16 ft.
Horse-power = 650 (325 h.p. per runner).
Revolutions per minute = 128.

These conditions, when reduced to a basis of 1 ft. head, give:

$$N_u = 32.5 \sqrt{5.07} = 72.4.$$

He refers to five tests submitted, but the writer will discuss only two—Nos. 1 and 4. These wheels showed the following values:

$$\text{No. 1}: N_u = 32 \sqrt{4.85} = 70.$$
$$\text{No. 4}: N_u = 35.7 \sqrt{4.00} = 71.4.$$

Wheel No. 1 was accepted and No. 4 was rejected, although the unit speed of the latter was nearer the desired value than that of the former. Mr. Mead justifies his choice on the ground that No. 1, while slightly deficient in power and speed, had high efficiency, whereas No. 4 had too high a value of N' (35.7) and too low a value of P' (4.00) and "was therefore unsuited to the conditions."

The writer takes exception to the last part of this statement, for the reason that wheel No. 4, although not itself of the proper size for this installation, nevertheless represents a type of wheel a little better suited to the conditions than No. 1. He does not criticize Mr. Mead's selection of wheel No. 1, because there is very little difference between the unit speeds of these two wheels, and, since it was possible to reduce the power of the units, wheel No. 1 fitted the conditions almost exactly both as to type and size. If, however, it had been essential to get the full 325 h.p., then (neglecting consideration of efficiency), the writer maintains that No. 4 was at least as suitable, if not more so, than No. 1, because, whereas both wheels would require alteration, No. 4 approximated more closely to the required type than No. 1, and was

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better evidence of the builder's ability to fulfill the specifications. The difference in this case is so slight that the distinction is purely academic, but, nevertheless, as illustrating a principle, No. 4 should have the preference, and the writer does not think that Mr. Mead is justified in stating that this wheel is not suited to the conditions. Assuming that No. 4 was a 41-in. wheel, the only change necessary to make it run at the right speed would be to increase its size. The new diameter would be:

$$D'_4 = \frac{41 \times 35.7}{32.5} = 45 \text{ in.}$$

As to the matter of standardizing runners, the writer thought that (on page 314) he had made his meaning clear, but Mr. Mead seems to have drawn mistaken conclusions. By "standardization" was meant the well-known practice of building several distinct types in various standard sizes and adhering to these standards even to the extent of using the wheels under conditions to which they are unsuited. By "special" wheels was meant the precise adaptation of the wheel to the conditions of operation, whether by the development of a new type or the modification of an old one, and it is specifically stated that the latter method embodies an element of greater reliability than the former. To quote from the paper:

"New designs will consist rather of modifications of and improvements on wheels which have been tested and found to be efficient, and, as such, will be more certain of success than if based wholly on theoretical considerations."

Mr. Mead, in the beginning of his discussion, speaks of the difficulty of designing wheels to develop their best efficiency under the fixed conditions of an actual installation. He says that he does not believe it is possible, without considerable experimental work, to design a wheel "that will unquestionably secure the desired results under actual operating conditions." The writer admits that the problem of making a wheel to fit fixed conditions, without any leeway, is the most difficult task the designer has to meet, but he believes that, with sufficient experience and a proper understanding of the work, it can be done without much uncertainty. Fig. 30 shows the efficiency curves of four runners of a new type designed by the writer for a set of fixed conditions. There was no experimental work of any kind preliminary to these four designs, which, although prepared to meet the same conditions, were radically different in their details.

Wheel 1801 was designed under the terms of a contract which required the wheels to show their maximum efficiency under conditions which, when reduced to a basis of 1 ft. head, were as follows:

Revolutions per minute = 28.3.
Discharge = 48.8 cu. ft. per sec.

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There is no more absolute way of specifying the performance of a wheel than that. Fig. 30 shows that the efficiency curve of this wheel peaked at exactly the specified discharge.

Inasmuch as there was a large bonus for efficiency at that discharge, the writer, being familiar with the loss of efficiency incidental to tests of large wheels at Holyoke, wished to find out whether the efficiency obtained with the first wheel could be improved upon. For this purpose, three more designs were prepared simultaneously, and as much variety as was feasible was embodied in these three additional wheels. They were purposely made different from the wheel that had been tested in order to determine if it could be improved upon. Fig. 30 also shows the tests of these three additional wheels, 1828, 1838, and 1839. As may be observed, the efficiency curves of all these wheels show practically their maximum values at the specified discharge.

The writer believes that if results as precise as these can be obtained four times in succession under conditions as stringent as stated, it must appear that no very considerable element of uncertainty exists.

With reference to Fig. 20, which Mr. Mead presents as evidence in support of his contention, the writer has no hesitation in saying that this is an example of inexcusably bad design. There was considerable leeway in the guaranty, and the design of the wheel was a simple matter. The test, however, shows that the wheel was grossly over capacity.

A great many such instances could probably be cited from the general experience of water-wheel users, but they do not represent by any means the best that is being accomplished in turbine construction in America at the present time.

In reply to Mr. Zowski's criticism relating to the omission of detailed descriptions of the wheels and other parts of the turbine, the writer would point out that the paper was necessarily limited in its scope. It was addressed particularly to turbine users, rather than turbine designers, and the former are not generally interested in details such as asked for by Mr. Zowski.

The writer attributes the good results obtained to the correct design of the runners and guides and the proper co-ordination of these parts. The mechanical work was no better than usual, and, in fact, the writer has very little faith in the efficacy of putting a high finish on the runner and guides. It is true, of course, that the smoother the walls of the water passages, the lower the friction loss will be, but the passages through the runner and guides are so short that the loss due to skin friction alone is very small in any case. The principal losses are due to internal friction or disturbance. In the case of these runners, the only finishing done on the vanes consisted in chipping off the fins caused by the clearance between the cores. The guide-vanes were finished only on their points of mutual contact.

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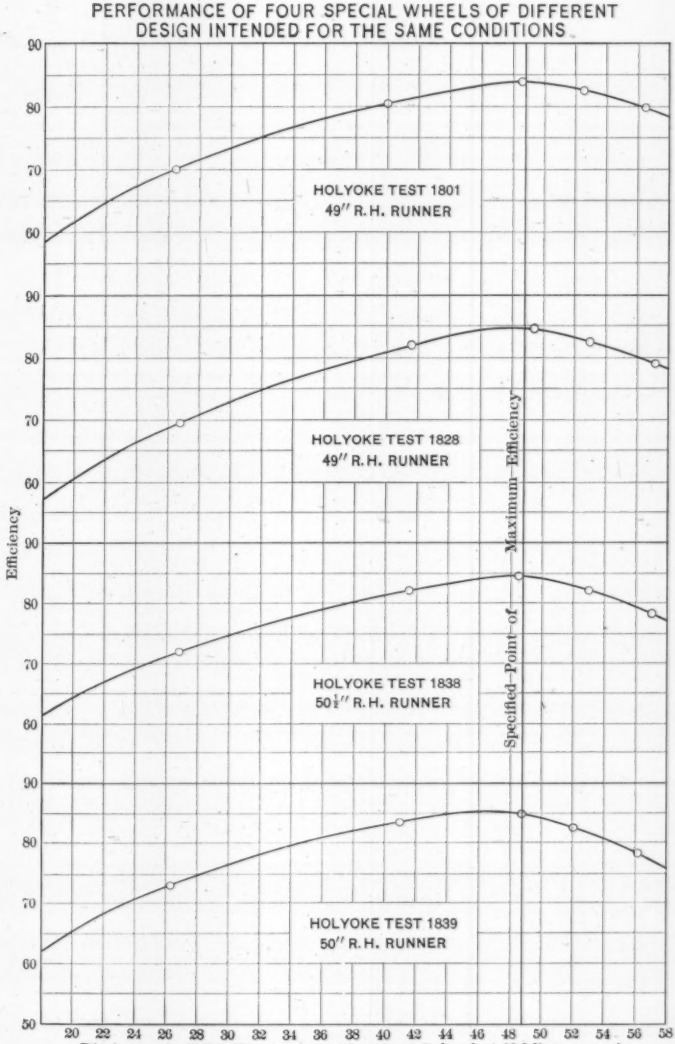

FIG. 30.

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Mr. Zowski refers to the sudden turn of the water in flowing from the guides into the runner, as shown by Fig. 1. If one imagines the water to flow over the edge of the curb plate in a radial direction, the condition would seem to be very bad, because the stream would have to turn a sharp right angle, but, if one considers the true course of the stream, which approaches this point in an approximately tangential direction, it is plain that the turn is not a sharp one. The water crosses this edge with a spiral motion which cannot cause any great disturbance of the stream.

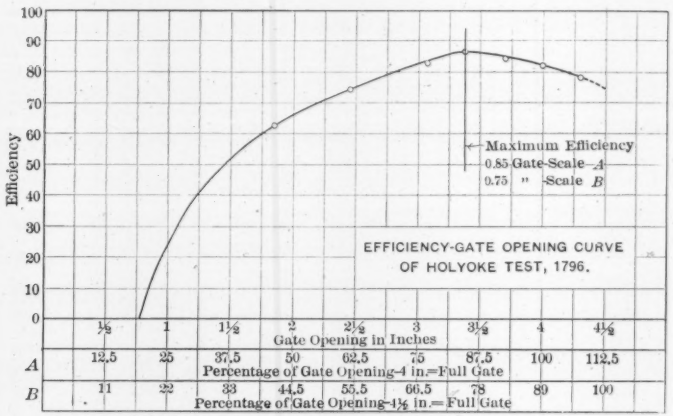

FIG. 31.

In regard to the European classification of runners, the writer has observed in many foreign textbooks that the value of ϕ is frequently used for this purpose. Attention was called to this fact in support of the contention that such a classification is not always consistent and never should be replaced by the use of the unit-speed method, which is exact. It was not claimed that the latter method is unknown in Europe. The writer was well aware that the method originated there and the European symbol, N_s, is referred to in the paper.

The writer takes decided issue with Mr. Zowski on the last statement in his discussion, namely, that "it is in the hands of the designer to have also with high-speed turbines the best efficiency at 0.75 gate, if this is preferable." Experiments and theoretical considerations show that this is an impossibility if the opening which is termed "full gate" is selected with reasonable regard for ordinary operating conditions. It can be done in the case of typical high-speed wheels only by over-gating the wheel to a point which would never be used under operating conditions.

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Fig. 31, showing Test 1796 plotted to efficiency and gate opening, will serve as an illustration. In reporting this test a 4-in. gate opening was selected as full gate for the reason that, as shown by Fig. 32, the wheel gave practically its maximum power at this gate. Any further opening would be over-gating, because considerably more water would be used to obtain a very trifling increase of power. In fact, if the gates had been opened to 4½ in., there would have been a loss of power rather than a gain. Under such circumstances, 4 in. should certainly be regarded as "full gate."

On the basis of 4 in. = full gate, the maximum efficiency occurs at 0.85 gate, as shown by Scale *A*. On the basis of 4½ in. = full gate, it occurs at 0.75 gate, as shown by Scale *B*. The latter assumption accords with Mr. Zowski's contention that the designer may make the maximum efficiency occur at 0.75 gate, but since it can be done only by over-gating to an extent which would never be used in practice, the assumption is untenable, from the standpoint of the operating engineer.

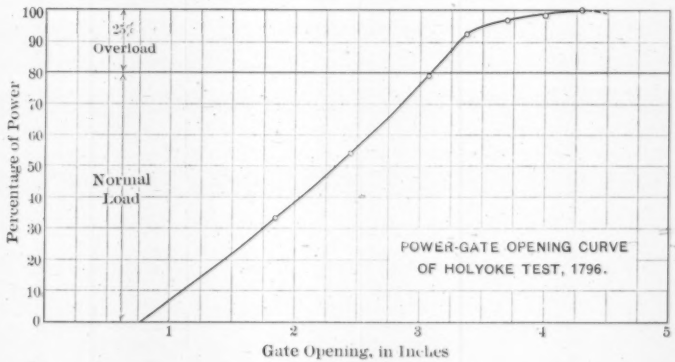


FIG. 32.

The writer regrets that he introduced the question of gate opening in his paper in this connection, because it is too indefinite a term to be of much value. One man will call a certain opening "full gate," and another will call some other opening "full gate." There is no way to determine the point definitely except to define "full gate" as the opening at which the wheel shows its maximum power. This method is open to the serious objection that in practice the gate opening is never, or very rarely, carried to this extreme point, because, by the time the peak of the power curve is reached, the efficiency will be relatively poor.

Percentage of load is a much more rational basis for performance guaranties than percentage of gate opening. Full power is readily

Mr. Larner. determined, whereas, full gate is not. Fig. 32, for example, shows plainly that the logical position of maximum gate opening is about 4 in. Also that "normal gate" is about 3 in. or 0.75 gate, and a comparison with Fig. 31 shows that maximum efficiency occurs at 92.5% load, or considerably above normal load. This is the point brought out in the paper, that with high-speed wheels the point of maximum efficiency is too near the peak of the power curve to give the best results, if the plant is designed for 25% overload.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1139

RIVER PROTECTION WORK ON THE KANSAS CITY SOUTHERN RAILWAY, NEAR BRADEN, OKLA.*

By J. A. Lahmer, M. Am. Soc. C. E.

With Discussion by Messrs. Charles H. Miller, and J. A. Lahmer.

When the branch of the Kansas City Southern Railway to Fort Smith, Ark., was built (in 1898), the track was about ¼ mile from the nearest point on the bank of the Arkansas River, near Braden, Okla. In recent years the river has been eroding its south bank at this place, so that in July, 1908, after the spring rise, it was 330 ft. from the track to the river. In the fall of that year there were unusual floods which caused the cutting of the bank to continue until the situation became very threatening, the river being within 180 ft. of the center of the track early in December, and erosion proceeding at a rapid rate. On account of topographic conditions it was not advisable to make a permanent change in the location of the track which, for some distance in this vicinity, is on the highest ground between the Arkansas and Poteau—one of its tributaries—which is a little less than a mile south of the track and nearly parallel to it.

The danger of a wash-out was averted temporarily by rolling into the river large cylindrical bundles of brush, 6 or 8 ft. in diameter and from 50 to 150 ft. long, one end of each bundle being anchored to the track by cables. These bundles of brush contained a layer of

* Presented at the meeting of November 3d, 1909.

sand bags, which caused the brush to sink. Next to the bank the current, of course, was very strong, so that the brush remained against the bank. In this way a wearing surface which the water would not cut was made from a line which was afterward found to be near low-water mark to a line a few feet above the flood stage, along that part of the bank which was so close as to endanger the track. All this work was done while the river was at flood stage. For several days it seemed that it would probably be necessary to move the track temporarily, but the brushwork proved equal to the emergency, and the river did not approach nearer than 50 ft. to the track.

Before the temporary work was completed, preparations were made for more durable protection work; in fact, plans had been adopted before the occurrence of the unexpected rise of December, 1908. The David Neale system of brush dikes had been selected, and bids were invited from contractors, but, on account of the scarcity of suitable material and the threatening condition of the river, no bids had been received. It was arranged, therefore, to do the work by Company forces under the direction of the Engineering Department.

The detailed plans of the dikes are shown on Plate XXXIV. Each dike consists essentially of a foundation of fascines, on which two or more stories or tiers of mud-cell rafts are built, as shown in the longitudinal section and plan, Figs. 1 and 2, Plate XXXIV.

The base of a fascine is made of a bottom layer of poles, *E*, from 4 to 6 in. in diameter and from 18 to 30 ft. long, on which there is a a layer of brush, *F*, 1 ft. thick. The brush and poles are laid longitudinally, with the butt ends of the brush outward at the ends of the fascine. Cross-walls, *G*, and side-walls, *H*, of brush are built on this floor in alternate courses from 10 to 12 in. thick, so that the fascine is divided into a series of cells from 6 to 8 ft. square and from 6 to 8 ft. deep. The length of the fascine was made equal to the width of the dike. The top of the fascine is a layer of brush, *I*, 1 ft. thick. The brush, in the walls and the top layer, is placed with the butt ends outward at the ends of the fascine.

An anchor of 6-in. poles, about 10 ft. long, is built in the second cross-wall from the up-stream end of the fascine. A $\frac{3}{4}$-in. or $\frac{7}{8}$-in. steel or iron cable (usually second-hand), secured to a post or tree on shore, is fastened to this anchor. Only every third or fourth fascine is anchored to the shore in this way, the intermediate ones being tied

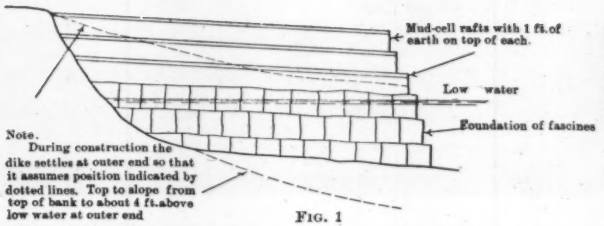
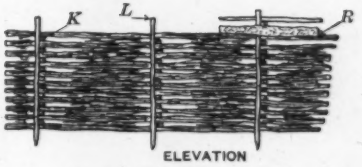
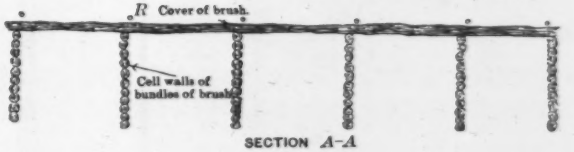


FIG. 1



ELEVATION

FIG. 7



SECTION *A–A*

FIG. 8

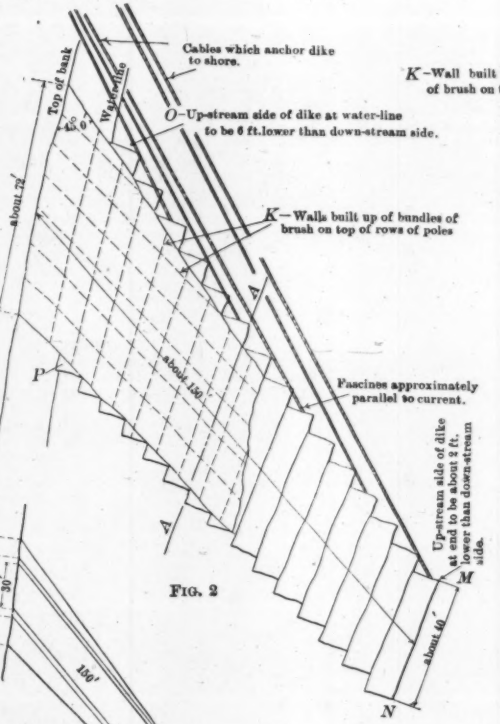
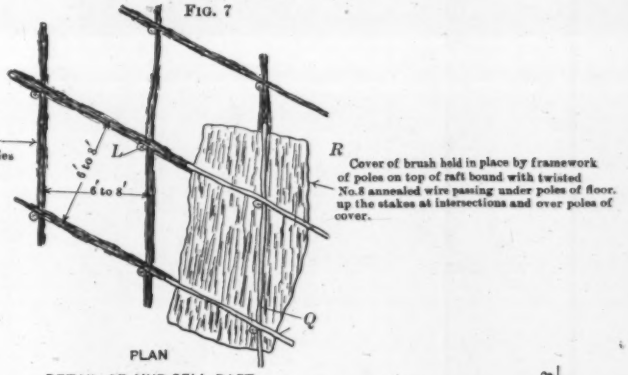


FIG. 2



PLAN

DETAIL OF MUD-CELL RAFT

FIG. 6

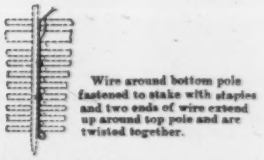


DIAGRAM SHOWING TIE WIRE AT WALL INTERSECTIONS OF MUD-CELL RAFTS

FIG. 9

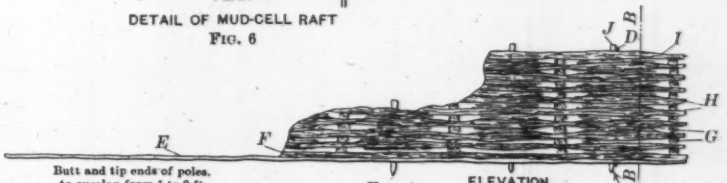
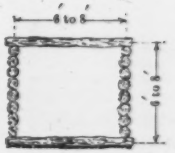
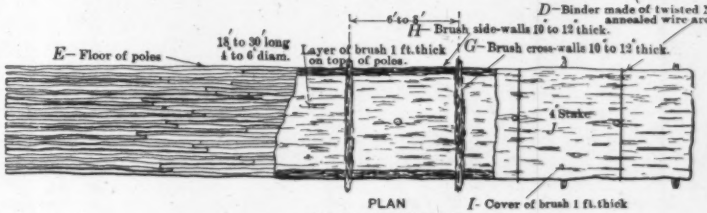


FIG. 4 ELEVATION



SECTION *B–B*

FIG. 5



PLAN

DETAIL OF FASCINE

FIG. 3

PLAN OF BRUSH DIKES

AS CONSTRUCTED ON THE ARKANSAS RIVER NEAR BRADEN, OKLA. FOR THE KANSAS CITY, SOUTHERN RAILWAY

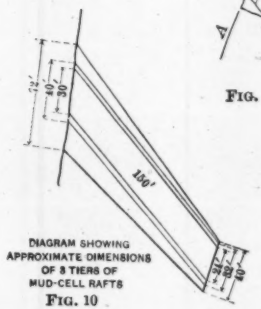


DIAGRAM SHOWING APPROXIMATE DIMENSIONS OF 3 TIERS OF MUD-CELL RAFTS

FIG. 10

by short cables to one of the cables which extends to the bank. Each cell is tied at the middle by a binder, *D*, made of three strands of No. 8 annealed wire. This binder is tightened by having its slack wound around a 4-in. stake, *J*, driven vertically through the center of each cell.

The first few fascines are built on the bank of the river and rolled into the water, until there are enough to form a space on which a fascine can be built and from which it can be rolled to the position it is to occupy in the dike. The work is continued until a foundation of the required width and length has been provided for the dike, the height of the foundation being such that the upper surface of the upper layer of fascines is above water level (Fig. 1, Plate XXXIV). Care is taken to place the fascine approximately parallel to the thread of the current, and to set each one a little down stream from the preceding one, in order to form an angle of approximately 45° between the thread of the current and the up-stream line of the fascine (Fig. 2, Plate XXXIV).

The lower mud-cell raft is started by laying rows of poles, from 4 to 6 in. thick, on top of the fascine foundation, lengthwise and crosswise of the dike, and from 6 to 8 ft. apart, as shown at *K* in Figs. 2 and 6, Plate XXXIV. The poles are wired together where they cross and are also wired to the binders of the upper course of fascines. At each intersection of the poles a stake, *L*, from 4 to 6 in. in diameter and from 8 to 10 ft. long, is nailed in a vertical position. A tie, of No. 8 annealed wire (Fig. 9, Plate XXXIV), is placed under the poles at each intersection and passed up along the stake, the ends of the tie extending about 1 ft. above the top of the stake. Walls of brush, *K*, are built in 10 or 12-in. courses along the rows of poles and on the up-stream and shore sides of the stakes, the courses being alternated lengthwise and crosswise of the dike until the walls are as high as the stakes. Fig. 8, Plate XXXIV, is a cross-section of the mud-cell raft.

The heights of the walls are regulated so that the finished dike at the outer end will be about 2 ft. lower on the up-stream side, *M*, than on the down-stream side, *N*, and, at the shore end of the dike, the up-stream side, *O*, will be about 6 ft. lower than the down-stream side, *P*, the object being to give the top of the dike a plow-share shape, in order to deflect the current as much as possible.

A cover of brush, *R*, 1 ft. thick, is built over the raft, the brush being placed approximately parallel to the thread of the current, with its butt ends outward at the up-stream and down-stream edges of the dike. On top of the brush cover, rows of poles, *Q*, are laid so as to intersect at the vertical stakes, the ends of the wires which were brought up the stakes being tied around the poles at the intersections, as shown in Fig. 9, Plate XXXIV. The wires are then twisted tight, so that the upper and lower framework of poles and the intervening brush will be bound together.

Each mud-cell raft is covered with a layer of earth, from 10 to 12 in. deep, care being taken to leave the poles exposed at their intersections. The wires which unite the poles at the bottom and top of the raft are again tightened after the earth has been deposited, as the weight of the earth will have compressed the brush.

The upper mud-cell rafts are built in manner similar to the lower one, except that the top of the lower raft is used as the bottom of the next raft above.

The lower mud-cell raft is built to cover as nearly as practicable the entire upper surface of the fascine foundation. The second and third stories of mud-cell rafts are narrower, the approximate dimensions of the three stories being shown in Fig. 10, Plate XXXIV. The dikes are built so that the outer end (usually about 150 ft. distant from the shore connection) will be 4 or 5 ft. above low-water mark, and the shore end either level with the bank or a few feet above high-water mark. The distance between the dikes is made as near 500 ft. as the condition of the bank will permit.

During a rise in the river, while sediment is being carried by the water, the dikes obstruct the flow, and, as the velocity is reduced, sand and mud are deposited in the hollow spaces of the fascines and mud-cell rafts. Sedimentation also takes place between the dikes, first on the up-stream side of the dike, near the river bank, and then on the down-stream side for a greater distance from the bank.

In several cases, notably Dikes 2 to 5, which were built first, considerable settlement occurred after the completion of the lower mud-cell raft, caused, no doubt, by the water which was deflected by the dike cutting under its end, or, perhaps, also by the weight of the dike forcing it down into a bed of quicksand. The number of stories of

mud-cell rafts on the several dikes is given in Table 1, which shows also the order in which the dikes were started.

TABLE 1.

Dike No.	Number of stories.	Dike No.	Number of stories.	Dike No.	Number of stories.	Dike No.	Number of stories.
2	5	1	2	*C*	2	*E*	1
3	5	*A*	2	6	2	7	2
4	4	*B*	3	*D*	2	8	2
5	3						

Three dikes were in process of construction at a time, and work was continued on each until two stories of mud-cell rafts were completed, when the dike was laid by. Later, as required, in order to keep the top of the dike at the proper height, additional stories were put on. After the December flood receded, there were, fortunately, no dangerous rises until several of the dikes had been laid by, but several rises of from 5 to 10 ft. occurred before the completion of any of the dikes. This was of advantage, in that it tended to fill, at least, the lower portions of the dikes and to collect enough deposit around them to increase their stability and add materially to their resistance to more serious floods. Work was begun in the middle of December, 1908, and the dikes were practically completed by the middle of April, 1909, but a small force was kept on the work until July, in order to have an organization which could be readily enlarged if made necessary by the action of the spring rise.

The water in the river rose to an elevation of 21½ ft. above low-water mark during the latter part of May, and remained high for two weeks. At ordinary low stage, there are from 16 to 18 ft. of water in the deepest part of the channel. The dikes, from Dike 2 up stream, were submerged. The south bank, from Dike 3 down stream, was above high water, but Dikes 3 to 6, inclusive, were under water, excepting 5 or 10 ft. at the shore ends. Dikes 7 and 8 were out of water for practically their entire lengths. The action of the dikes, as a protection to the bank, was reassuring, for none of the bank was lost, except at two places, and there the loss was inconsiderable.

Just below Dike 3, a slice of the bank, 6 ft. wide and 50 ft. long, caved off the day before the crest of the flood was reached, after which no trouble was experienced. Between points 50 and 110 ft. below the

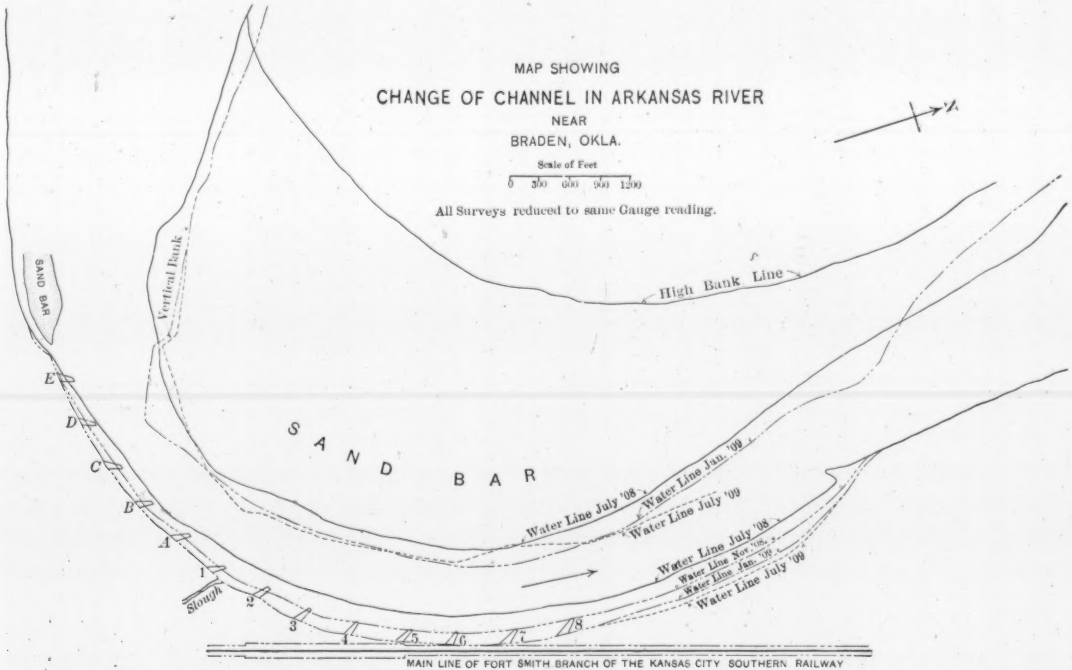

MAP SHOWING
CHANGE OF CHANNEL IN ARKANSAS RIVER
NEAR
BRADEN, OKLA.
Scale of Feet
0 300 600 900 1200
All Surveys reduced to same Gauge reading.
SAND BAR
Vertical Bank
High Bank Line
SAND BAR
Water Line July '08
Water Line Jan. '09
Water Line July '09
Water Line July '08
Water Line Nov. '08
Water Line Jan. '09
Water Line July '09
Slough
MAIN LINE OF FORT SMITH BRANCH OF THE KANSAS CITY SOUTHERN RAILWAY

FIG. 1.

down-stream side of Dike 5, the bank began falling off on the day the water reached its highest stage. A total width of only 8 or 10 ft. was lost, but several solid fascines were launched before the caving stopped. While the river was rising, a strong current was deflected around the end of each dike, so that it struck the outer half of the next dike down stream. A weaker current turned up stream from the dike and flowed backward, usually close to the shore, until it struck the next dike up stream, from which it was deflected and flowed down stream—until its energy was spent—along a path about midway between the shore and the line along which the current had previously gone down stream. At the flood stage the same action took place in a general way. Sometimes the back current flowed up stream only half way to the next dike before it was deflected down stream by a direct current, and more violent eddies were produced where the strong down-stream currents, which passed over and around the end of a dike, collided with the quieter water below the dike and nearer the shore. The eddies which were produced below Dikes 3 and 5, were so close to the shore—within 30 or 40 ft.—that their energy was not spent before they impinged on the slope of the bank with sufficient force to bore holes in it. It is the writer's opinion that damage of this kind by future floods could be prevented by raising the half of the dike next to the shore, and this has since been done on Dikes 3, 4, and 5.

Since the water has returned to low stage, sand-bars have appeared along the shore between the dikes. These bars have a width (transverse to the stream) of from 20 to 40 ft., and elevations of from 2 to 5 ft. above low-water mark. The cells in the dikes are filled to the same elevation, the deposit extending 5 or 6 ft. outside the dike, except along the outer end and along the outer half of the dike on the up-stream side. In nearly every case the deepest water in the river is just beyond the outer end of the dike.

There is no evidence that any cutting has taken place on the north side of the river, opposite the dikes. Erosion continued on the north bank of the upper end of the large bar shown on the map, Fig. 1. It seems, therefore, that the dikes must take care of the south bank until this erosion progresses far enough down stream to wear off the bar. The south bank, east of a point about 400 ft. below Dike 8, was cut off as much as 75 ft. in places.

The principal damage sustained by the dikes was to their looks. The

outer ends of the upper rafts of Dikes *D* and 3 were washed out of line, so that they hung over the down-stream sides of the next lower rafts, but the upper rafts were not broken apart; and the outer ends of Dikes 2 and 3 were twisted out of shape, but were not detached. Nearly all the dikes settled a little at their outer ends, but none of this damage is sufficient to impair their efficiency. The portion of the upper raft of Dike 5, between 70 and 100 ft. from the shore connection, was torn loose and washed away, and will have to be replaced. This is the only part that was lost. The ability of the dikes to withstand the force of a flood has been established beyond reasonable doubt.

Willows, up to 2 in. in diameter at the butts, are most desirable for constructing the dikes, but, on account of the lack of this kind of material within easy reach of the work, it was necessary to use also pine and hardwood brush. Willows were cut and tied in bundles, 10 or 12 in. in diameter, at Kansas City, and shipped to the site of the work. This supply was limited and, before the work was completed, was shut off by water in the Missouri River bottoms. The pine and hardwood brush could not be tied in bundles and was more expensive and inconvenient to handle than willows, besides causing a less satisfactory grade of work. A total of 18 000 cords of brush and poles (loose measurement, except in willows) was used in the permanent work, and the average cost was $1.55 per cord on board cars at the site of the work. Less than one-sixth of the total amount of brush was willows. No charges were assessed for transportation of material over the Company's line.

The cost of constructing the dikes amounts to $8.15 per ft. of bank protected, counting from the up-stream connection of Dike *E* to a point 400 ft. below the down-stream connection of Dike 8. The economy of this system of work will depend largely on the cost of maintaining the dikes until such time as they will be no longer required for the protection of the bank, which cost is as yet unknown. The portions which are below water will last a long time, as far as decay is concerned, but the portions which are above the sediment collected by the dikes will have a much shorter life. Data as to the durability of the exposed portions of the dikes will be awaited with interest.

DISCUSSION

Mr. Miller.

CHARLES H. MILLER, M. AM. SOC. C. E. (by letter).—Having visited this work several times, and having become familiar, to some extent, with the David Neale system during the past three or four years, the writer has read with much interest Mr. Lahmer's detailed description.

When the writer began the investigation of this system, several years ago, he was confronted with the facts that there were no accurate cost or location records covering the work which had been placed, some of it, as many as twelve years before; that often there was no way of telling whether or not the changes occurring in the vicinity were brought about by the influence of the dikes or through natural causes; that some of the dikes seemed to have served well the purpose for which they were constructed, while others were found to have been entirely flanked and dropped into the bed of the river; that is, the river got behind the dike and continued to scour away the bank, until, in some cases, the dike was left on the opposite or bar side of the river.

In most cases it developed that no repairs or additions were made after the work was first constructed, it being the contention of the inventor that after it once became covered with sediment, it would be practically permanent.

One point, however, seemed to come out prominently in most cases, namely, that the dikes possessed an inherent strength against being broken up piece by piece and carried off by the currents, and although entirely cut down by scour underneath and behind, and much bent and twisted, the main mass, especially that under the low-water level, remained, those parts above low water sustaining losses of one or more layers due to the decay of some of the materials composing them.

With this quality in view, this system was adopted, under the assumption that sufficient repairs could be made from time to time to hold the dikes in effective position, and, in the end, that a protection would be obtained at less cost than for a continuous system.

At Myrick, Mo., on the Missouri River, dikes have been in place for three years, and their position is being maintained, but this is much too short a period in which to obtain a reliable comparison between this and other systems. It has developed that an error was made in placing these dikes too far apart, and it has been necessary to build short pieces of bank revetment, both above and below them, for distances of approximately 100 ft., in order to prevent them from being cut down. The original cost of this work was $2.50 per lin ft. of bank; the repairs and additions placed amount to $1 per lin. ft., making a total of $3.50 per lin. ft.

This system does not contemplate any particular size or shape of dike, but fits the physical conditions as well as the class of materials

Mr. Miller. to be found in the locality, and, naturally, there is a difference of opinion as to the most effective shape. The writer now uses a form which differs very much from that described by Mr. Lahmer. It is triangular or pyramidal in shape,* and the main part does not jut out into the stream as much as the long and narrow dike described in the paper. This triangular dike is now modified by discarding the large fascines for the foundation, and simply using a mattress, either of the solid or of the mud-cell type; it having been demonstrated clearly that the fascine method is the more expensive, first as to its actual construction, and second, because it permits more scour under the edges, hence more settlement, and, consequently, more material is needed to construct the dike to the required height. The mattress foundation is constructed so as to extend out beyond the edges of the proposed dike, and, as the scour progresses, this extended portion of the mattress, being pliable, drops and prevents excessive underscour. The mattress, extending a short distance above and below the dike, also prevents the eddy, which cannot be entirely overcome, from cutting behind the upper bank part of the dike.

In calculating the relative effectiveness of the different forms of dikes much depends on the height and duration of floods, but the writer is firmly convinced that for the Missouri River, where floods often remain at bank-full stage for a month or two at a time, the form of dike described by Mr. Lahmer will not hold up as well as it will in the Arkansas River, where the floods are usually of much shorter duration, and that it will be necessary to place some top-bank protection both above and below the dike. In other words, the extent of the dike, up and down stream, at the top of the bank, will prove to be insufficient for a long period of high water. The dike described is intended to throw the main body of the river so far away from the main bank that the influence of any eddies which may develop will not be felt at the bank. This will be found to be true for all ordinary stages of the river, at which times a considerable portion of the dike is above the water surface, and the water is forced around it at some distance from the general shore line or edge of the water. This distance gradually diminishes as the river rises, and when it is high, the water flows over the dike only a few feet out from the general shore line, and there is then nothing to prevent it from scouring out under the edge of the dike.

It is true that the stronger force of the currents is farther out and lower down from the top bank; but there is often sufficient force along the top bank during high water to cause much damage to such work. Mr. Lahmer calls attention to a caving, from 8 to 10 ft. in width, below Dike 5. It is just these small amounts at each recurrent flood which will soon result in the dike being on a narrow strip jutting out from

* Described in detail and illustrated in *Engineering News*, October 22d, 1908.

the main bank, and this strip is usually quite susceptible to being cut out by eddy currents, thus allowing the upper part of the dike to settle down, thereby giving the currents freer play and a chance to scour more rapidly. Mr. Miller.

The necessity for extending the system of dikes or protection work such a great distance up the "bend" is not quite clear, in fact, it is believed that it would have been better not to have done so, at least during the first year. The best information at hand discloses no caving in this part of the river of an amount greater than about 300 ft. per annum; it is usually very much less; and, therefore, it is safe to predict that the railway tracks would have been perfectly safe for at least one year more, if no work had been placed above Dike 2, which is about 500 ft. from the track. Then, after the caving had continued so as to form a "point" in the "bend" at Dike 2, one or two additional dikes could be built each year, as the scouring progressed up stream, the net result being that new work would occupy the position of greatest attack as it moved up stream, and that the "false point" would tend to deflect the main river below the same so as to relieve the lower and older dikes. In fact, it is highly probable that it would deflect it so as to cause the sand bar to scour off to such an extent that the river would give no further trouble below Dike 8, and that by the time Dikes 2 to 8 became dilapidated from age they would no longer be needed.

The form of the "bend," as now protected, is that which usually obtains in this part of the river, and it is quite likely to remain so for many years; which means that all these dikes must be maintained continuously, also that the tendency of the river will be to cave in gradually and straighten out below Dike 8, thus necessitating additional work to protect that part of the railway track.

If the plan to stop the upper end of the work with Dike 2 had been adopted, this dike, being in a very exposed position, would no doubt have been damaged considerably and possibly cut down entirely; however, it would have been much reinforced on account of its exposed position, and in all probability would not have been entirely lost. In any event, the track could not possibly have been reached, the desired "point" would have been formed, and the conditions for this particular case much improved. It is the almost positive assurance of a loss of some extent, when the head of the protection work is left within the limits of active caving, which is objected to so strongly by many, and is the reason for extending the work to a point above the caving, as was done in the work at Braden.

It is hardly safe to depend on the erosion noted on the north bank of the upper end of the large bar at the point marked "vertical bank" on Fig. 1, nor to assume that eventually it will extend down so as to bring the river below the dikes, thus obviating the necessity of maintaining them, because this would be a very unusual manner for this

Mr. Miller.

caving to progress. Had Fig. 1 been extended to show the "bend" above this point, it is quite likely that it would have disclosed a greater rate of erosion or caving at some distance above the "vertical bank," tending gradually to throw the impact of the river away from the "vertical bank," and also tending to keep the river directed toward the upper dikes in the protection system.

On the other hand, if this unusual state or condition did occur, it would be one under which the dike system would be extremely hard to maintain, the one under which the continuous system of revetment or protection would be the most economical in the first place, because it would be necessary no doubt to place continuous protection between the dikes as they came within the line of almost perpendicular attack.

The dike system does not give the same security, in fact, as the continuous system, and where the bank is as near the railway as is the case in the vicinity of Dikes 6 and 7, careful and constant watching will always be necessary during each flood period.

With the entire "bend" protected, as it is now, no marked relief can be expected. In this case, however, the top bank is composed in large part of a hard clay, and, therefore, the progress of any caving which may develop when the river is high will necessarily be quite slow.

One reason for using the dike system is its economy as compared with the continuous system, brought about chiefly because the dikes deflect the currents so that part of the bank needs no protection; and much greater economies can be brought about when advantage can be taken of a "point" in a "bend" so that it can be held as an enormous dike.

Mr. Lahmer describes the use of large bundles of brush or hollow fascines as temporary protection against the rapid erosion just before the general work was started, and again to stop the caving just below Dike 5 during the flood which took place after its completion. It is for such emergency cases that these large fascines are especially useful, the writer having on several occasions successfully stopped very rapid caving with them. These fascines can be built on the river bank, and launched into place without any floating or special plant of any kind.

Any variety of brush and poles can be used, and the size may be quite variable; if the brush contains no leaves and small branches, then weeds, hay, or straw must be used to form tight sides and partitions. It is also possible to use sawed lumber instead of brush and poles. Telegraph wire may be used to bind the bundles or fascines together, and several wires can be twisted together to form cables or strands for anchoring the fascines to the bank. Some spikes can be used to advantage also. In cases where the river does not carry much sediment, and, in any event, it is well to have some empty sacks to be filled with soil and placed within the fascines to cause them to sink into place more quickly.

All or a sufficient quantity of these materials can usually be obtained readily and rushed to the point of danger by a work train. Large gangs of laborers can build one or more fascines simultaneously, and roll them into the river as fast as they are constructed. The stage of the river is immaterial, as long as the area of the bank or dry land on which the fascines are to be built, is large enough. Mr. Miller.

It is earnestly hoped that Mr. Lahmer will present figures showing the cost of maintenance.

J. A. LAHMER, M. AM. SOC. C. E. (by letter).—Mr. Miller states that this system of protection work was constructed on the Missouri River at Myrick, Mo., at a cost of $3.50 per lin. ft. (of bank protected). This cost per foot is very much less than the cost of the work at Braden, on the same basis, but a fair comparison between the cost of the two jobs cannot be made, because the distance between the dikes at Myrick is not given, the only information at hand being that they were placed too far apart. Mr. Lahmer.

The satisfactory results obtained by Mr. Miller, from the use of short dikes high on the shore end, seem to confirm the writer's opinion that the cutting of the bank just below the dikes will be prevented by raising the half of the dike which is next to the shore. It may be considered that, in the triangular or pyramidal-shaped dike, Mr. Miller omits a part, if not all, of the outer half of the dike as constructed at Braden. The writer's experience is not sufficient to enable him to decide just what value to place on this outer half, but there is no doubt in his mind that it is of considerable service in deflecting the current from the bank and also in helping to take care of the scour at the end of the dike.

It is very likely that in some instances a mattress foundation could be used profitably instead of fascines. The work at Braden was not built during the most favorable season, and the conditions were such that it would have been more difficult, and probably more expensive, to have used mattress foundations. It is assumed that the mattress would be made on the river bank and floated into place, or that some other method requiring barges and a power boat would be used. Fascines do not require such equipment. They can be made on the bank and rolled into the water, even though the bank be steep and the current swift; and, if now and then a fascine fails to land in the proper position no serious consequences follow, for due allowance can be made in placing the next few fascines.

The writer agrees with Mr. Miller regarding the work in the upper part of the bend of the river, and was in favor of postponing this part of the work until further developments could have taken place, but higher authorities considered that it would be better to extend the work to the upper end of the cutting bank at once. It was first intended

Mr. Lahmer.

to construct only those dikes which are numbered; the dikes designated by letters were added to the plans after the work was begun.

On account of lack of experience with this kind of work, the writer naturally tried to keep on the safe side, both as to the quantity of material used and the extent of the work. The quantity of material required for the first few dikes built at Braden was much greater than was expected. This was due in part to the high stage of the river at the time they were built, and perhaps, also, to the presence of quicksand in the bed of the river.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1140

PROGRESS REPORT OF SPECIAL COMMITTEE ON STEEL COLUMNS AND STRUTS.*

WITH DISCUSSION BY THOMAS H. JOHNSON, M. AM. SOC. C. E.

The Special Committee authorized by vote of this Society "to consider and report upon the design, ultimate strength and safe working values of Steel Columns and Struts" submits the following report of progress.

Your Committee met and organized on January 6th, 1909. Since that time four meetings of the Committee as a whole and numerous sub-committee meetings have been held. Time and labor have been spent in collecting existing literature on the subject of tests of compression members, in investigating and compiling the data now accessible, in a preliminary consideration of the extent to which it is safe to draw conclusions from data now available, and in a study of the possibility of securing additional information. At the last Annual Meeting, your Committee submitted to the Society a resolution requesting the United States Government to construct a machine capable of testing to destruction compression members of large dimensions.

Your Committee finds that there are at present available for use in this country but three testing machines of large size, while only two are capable of making compression tests. The data giving principal dimensions and capacity of the machines are submitted in Table 1.

These machines will be supplemented in the near future by two others. A vertical machine of 10 000 000 lb. capacity in compression, capable of testing columns up to 65 ft. in length is now being built for the Geological Survey. This machine will be located in Pittsburg. Under authority of the Congress of 1908-09, the Bureau of Standards has contracted for a horizontal machine with a capacity of 2 300 000 lb. in compression and 1 150 000 lb. in tension, of the same type that is now in use at Watertown Arsenal. This machine is to be located at Washington, D. C.

* Presented to the Annual Meeting, January 19th, 1910.

TABLE 1.—DATA RELATING TO TESTING MACHINES.

	Phœnixville, Pa.	Ambridge, Pa.	Watertown, Mass.
Date of completion	About 1886	1905	1879
Capacity for tension tests	2 600 000 lb.	4 000 000 lb.	500 000 lb.
Capacity for compression tests	2 600 000 lb.	none	800 000 lb.
Maximum length of test pieces:			
Tension	50 ft.	39 ft.	20 ft.
Compression	55 "	none	26 " 3 in.
Available clear width for tests	3 ft.	3 ft.	42½ in.
Power	Hydraulic	Hydraulic	Hydraulic
Recording device	Mercury gauges	Mercury gauge	Emery hydraulic scale
Height from floor to center of ram	About 4 ft.	2 ft.	47 in.

Your Committee has examined the records of numerous compression tests made on pieces varying in area of cross-section from laboratory specimens to 42 sq. in. While the results given by small specimen tests are interesting, your Committee has selected only the tests of columns large enough to be used as structural members, and these have been plotted on Figs. 1 to 22, inclusive. Fig. 23 gives a summary of results. Your Committee has also considered some tests which it is not now at liberty to publish, and it is expected that the data given in this report will be supplemented from time to time as these and other tests become available for publication.

In the use of $\frac{l}{r}$ (length divided by radius of gyration) as a variable, your Committee realizes that r may not be the only variable affecting the strength of a column of given length. Shape, details, and dimensions, may make the results very different in columns having the same value of $\frac{l}{r}$. Until many extended series of tests afford more information with reference to these elements, it is necessary to follow common practice in plotting.

To reduce the results of the various series of tests shown on the diagrams to a uniform basis, the values have been adjusted to a grade of steel of 60 000 lb. ultimate tensile strength in specimen tests, by assuming that the ultimate compressive strength will vary as the ultimate tensile strength. This method of adjustment probably does not accurately represent the law of variation of the different grades of iron and steel, still, as the adjusted value is a fair medium between the older iron and the newer steel, it affords some basis for comparison.

A casual examination of the plates shows a meager range of experiments, made on no systematic plan, and a striking disagreement of

results, even after adjustment for quality of material. It is evident that these data are not sufficient for the establishment of a satisfactory law for the strength of columns. This emphasizes the importance of further tests on columns arranged in series under conditions of material, workmanship and details, which will aid scientific observation in the establishment of the effects of the different variables.

At the time your Committee was appointed, circumstances caused a demand for tests to destruction of compression members equal in size to the largest ever built for existing bridges. This demand was backed largely by both the popular and technical press, but careful consideration and study lead to the conviction that while such tests are desirable, the cost of making them is so great that there is no chance of obtaining a sufficient number to disclose any fundamental law. Elaborate series of tests on smaller columns, in which the details and processes of manufacture conform to commercial practice, would disclose the effects of the different variables and would determine the fundamental laws of the behavior of columns under stress. Such a series of tests, with the necessary investigations, should properly be made by the Federal Government. For comparison with the results obtained from these series, tests of members, modeled after columns which are too expensive to allow more than an occasional test, and too large to be handled conveniently, would afford valuable data.

Your Committee recognizes the urgency of the demand for a statement giving values for ultimate strength and safe working stresses for compression members, but in their opinion existing data do not warrant the recommendation of any definite column formula. Until the subject can be studied more at length and until more extended series of tests have been made, your Committee does not deem it advisable to complicate the situation by offering a tentative formula which would vary but little from other formulas now in use. When the results of further tests become available for study and investigation, your Committee hopes to be able to make a recommendation for a column formula.

Committee.
Alfred P. Boller.
Austin Lord Bowman.
Emil Gerber.
Charles F. Loweth.
Ralph Modjeski.
Frank C. Osborn.
Geo. H. Pegram.
Lewis D. Rights.
Geo. F. Swain.
Emil Swensson.
Joseph R. Worcester.

For the Committee,
AUSTIN LORD BOWMAN,
Chairman.
LEWIS D. RIGHTS,
Secretary.

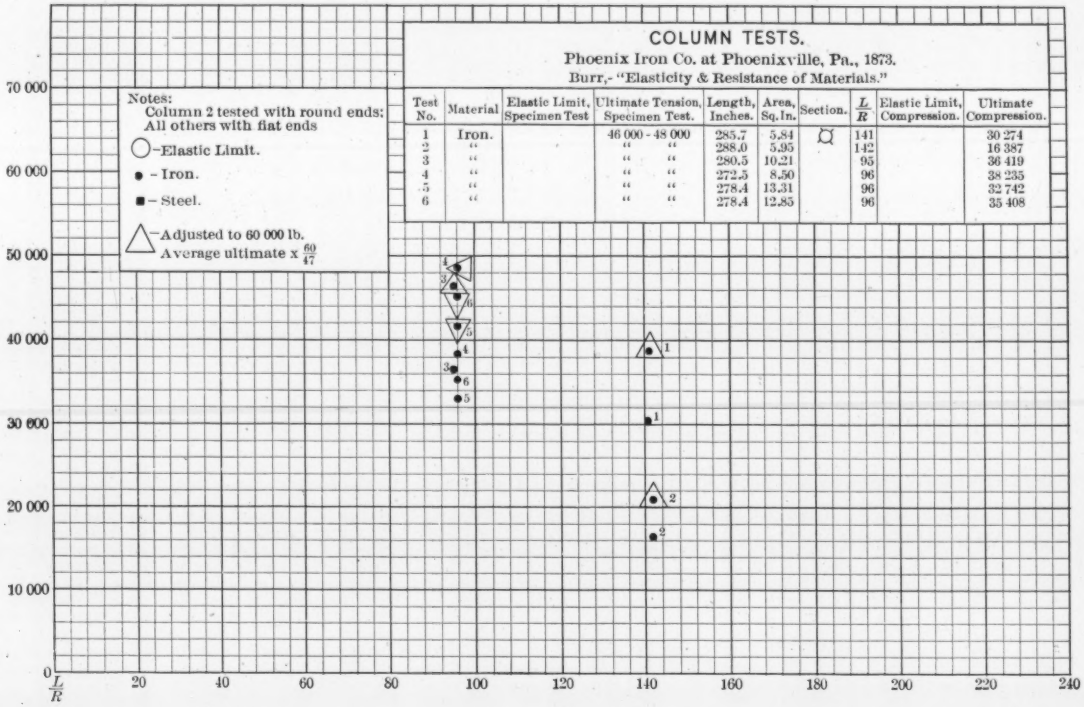

COLUMN TESTS.
Phoenix Iron Co. at Phoenixville, Pa., 1873.
Burr,- "Elasticity & Resistance of Materials."
Notes:
Column 2 tested with round ends;
All others with flat ends
-Elastic Limit.
- Iron.
- Steel.
-Adjusted to 60 000 lb.
Average ultimate x 60/47
Test No. | Material | Elastic Limit, Specimen Test | Ultimate Tension, Specimen Test. | Length, Inches. | Area, Sq. In. | Section. | L/R | Elastic Limit, Compression. | Ultimate Compression.
1 | Iron. | | 46 000 - 48 000 | 285.7 | 5.84 | | 141 | | 30 274
2 | " | | " " | 288.0 | 5.95 | | 142 | | 16 387
3 | " | | " " | 280.5 | 10.21 | | 95 | | 36 419
4 | " | | " " | 272.5 | 8.50 | | 96 | | 38 235
5 | " | | " " | 278.4 | 13.31 | | 96 | | 32 742
6 | " | | " " | 278.4 | 12.85 | | 96 | | 35 408
70 000
60 000
50 000
40 000
30 000
20 000
10 000
0
L/R
20
40
60
80
100
120
140
160
180
200
220
240

FIG. 1.

COLUMN TESTS.

G. Bouscaren.

Transactions, Am. Soc. C.E. Vol. IX, 1880.

Test No.	Material.	Elastic Limit, Specimen Test.	Ultimate Tension, Specimen Test.	Length, Inches.	Area, Sq. In.	Section.	$\frac{L}{R}$	Elastic Limit, Compression.	Ultimate Compression.
21	Iron.		52 000 - 55 000	309	13.600	I	93	18 000	25 500
22	"		" "	312	13.600	"	102	16 000	30 000
23	"		" "	288	13.700	"	84	15 000	33 200
32	"		" "	324	26.050	"	98	15 000	30 200
36	"		" "	408	7.480	[	138	21 600	23 128
37	"		" "	329.7	12.078	][	74		29 600
38	"		" "	276.1	13.476	"	61	23 000	32 300
41	"		" "	330	13.740	"	72	20 800	32 400
42	"		" "	330	11.052	"	72	21 000	32 300

Notes:
Columns 21 & 36 tested with Pin Ends;
All others with Flat Ends.

○ — Elastic Limit.

● — Iron.

■ — Steel.

△ — Adjusted to 60 000 lb.
Average ultimate x $\frac{60\,000}{53\,500}$

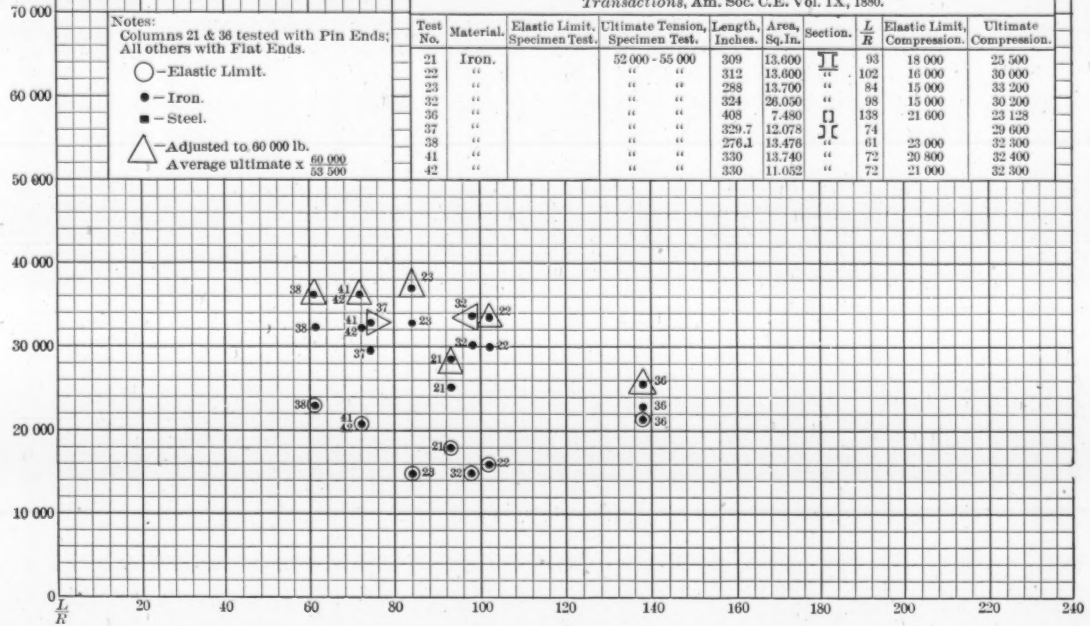


FIG. 2.

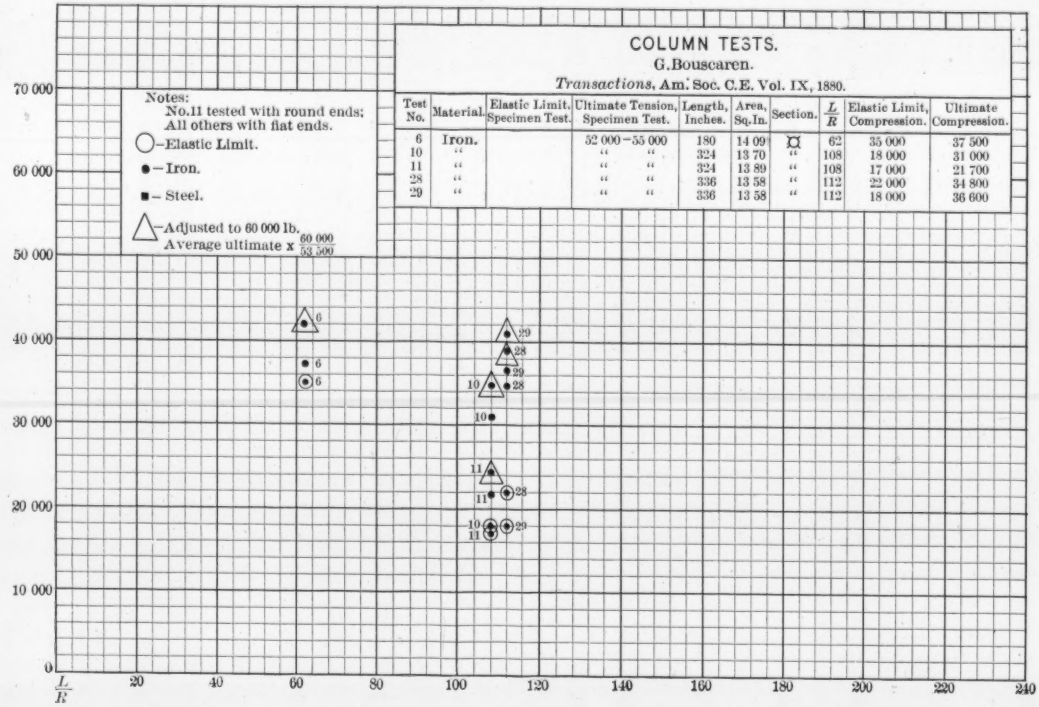
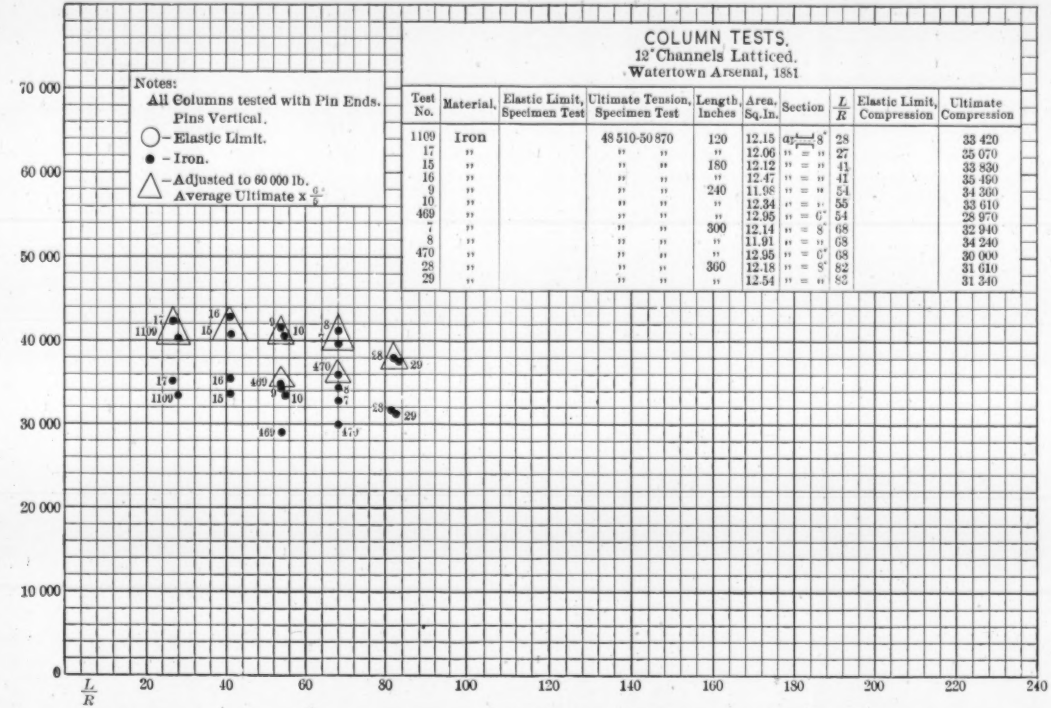

COLUMN TESTS.
G. Bouscaren.
Transactions, Am. Soc. C.E. Vol. IX, 1880.
Notes:
No. 11 tested with round ends;
All others with flat ends.
○ – Elastic Limit.
● – Iron.
■ – Steel.
△ – Adjusted to 60 000 lb.
Average ultimate × 60 000 / 53 500
Test No. | Material. | Elastic Limit, Specimen Test. | Ultimate Tension, Specimen Test. | Length, Inches. | Area, Sq. In. | Section. | L/R | Elastic Limit, Compression. | Ultimate Compression.
6 | Iron. | | 52 000 – 55 000 | 180 | 14 09 | ⌑ | 62 | 35 000 | 37 500
10 | " | | " " | 324 | 13 70 | " | 108 | 18 000 | 31 000
11 | " | | " " | 324 | 13 89 | " | 108 | 17 000 | 21 700
28 | " | | " " | 336 | 13 58 | " | 112 | 22 000 | 34 800
29 | " | | " " | 336 | 13 58 | " | 112 | 18 000 | 36 600
70 000
60 000
50 000
40 000
30 000
20 000
10 000
0
L/R
20
40
60
80
100
120
140
160
180
200
220
240

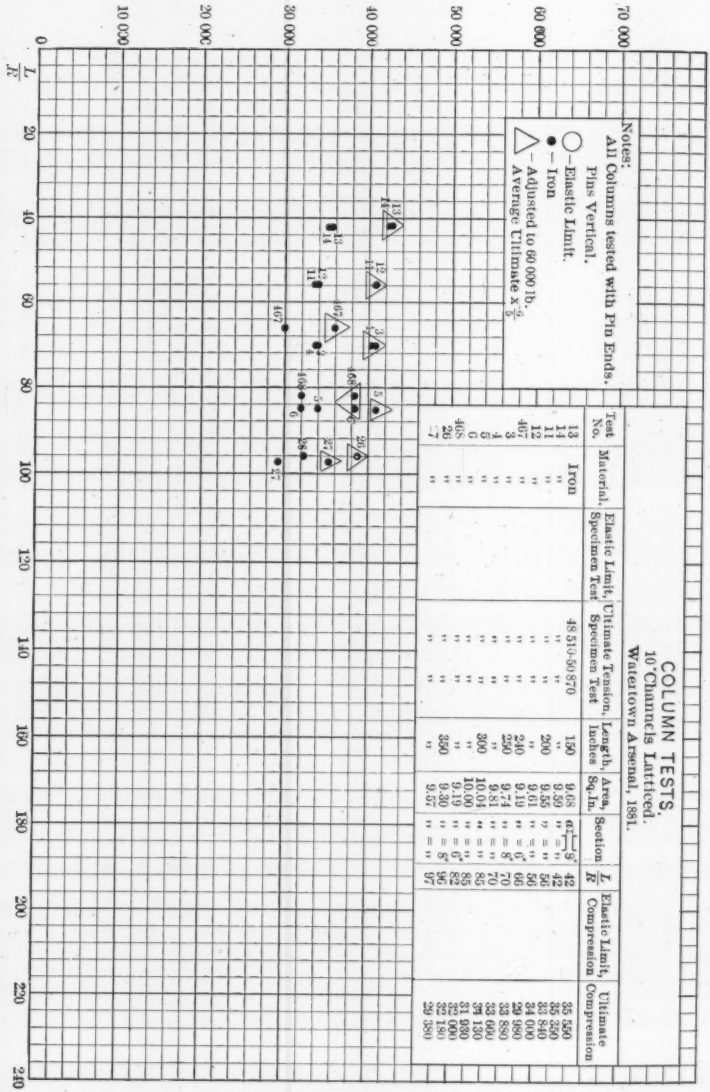
FIG. 3.



COLUMN TESTS.
12" Channels Latticed.
Watertown Arsenal, 1881

Test No.	Material.	Elastic Limit, Specimen Test	Ultimate Tension, Specimen Test	Length, Inches	Area, Sq.In.	Section	$\frac{L}{R}$	Elastic Limit, Compression	Ultimate Compression
1109	Iron		48 510-50 870	120	12.15	$a_{5}^{1}\sqrt{\square}8''$	28		33 420
17	,,		,, ,,	,,	12.06	,, = ,,	27		35 070
15	,,		,, ,,	180	12.12	,, = ,,	41		33 830
16	,,		,, ,,	,,	12.47	,, = ,,	41		35 490
9	,,		,, ,,	240	11.98	,, = ,,	54		34 390
10	,,		,, ,,	,,	12.34	,, = ,,	55		33 610
469	,,		,, ,,	,,	12.95	,, = 6"	54		28 970
7	,,		,, ,,	300	12.14	,, = 8"	68		32 940
8	,,		,, ,,	,,	11.91	,, = ,,	68		34 240
470	,,		,, ,,	,,	12.95	,, = 6"	68		30 000
28	,,		,, ,,	360	12.18	,, = 8"	82		31 610
29	,,		,, ,,	,,	12.54	,, = ,,	85		31 340

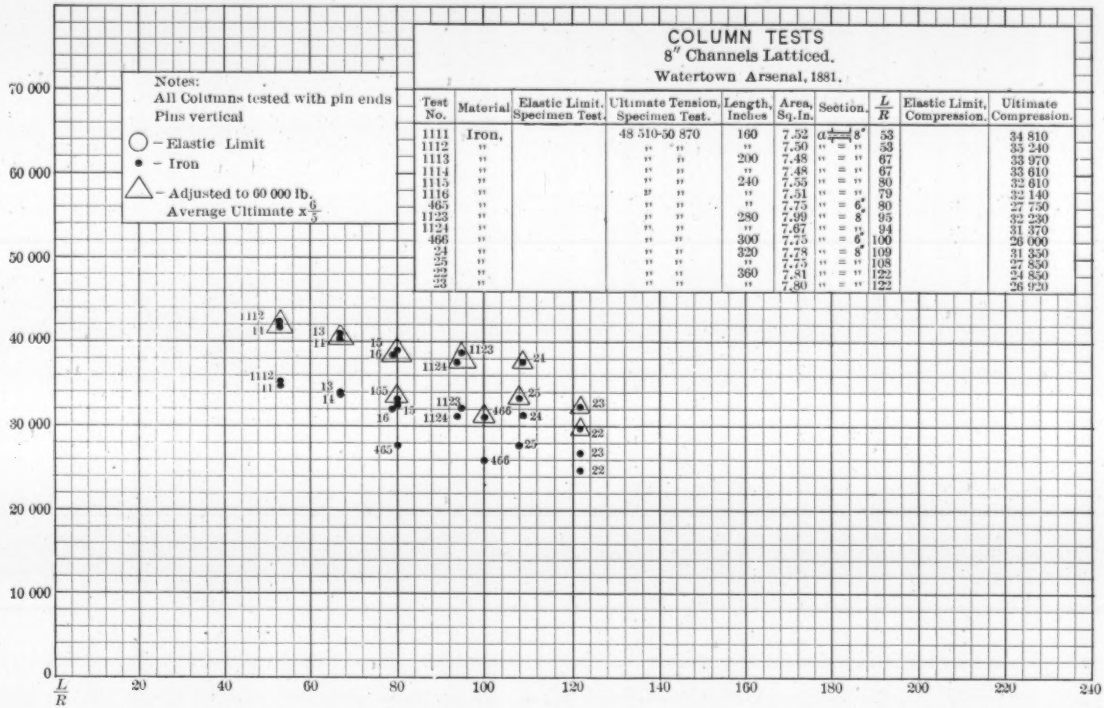
FIG. 4.



COLUMN TESTS.
10" Channels Latticed.
Watertown Arsenal, 1881.

Test No.	Material.	Elastic Limit, Specimen Test	Ultimate Tension, Specimen Test	Length, Inches	Area, Sq. In.	Section	$\frac{L}{R}$	Elastic Limit, Compression	Ultimate Compression
13	Iron		48 510-50 870	150	9.68	$\alpha = \frac{1}{2}$ 8"	42		35 550
14	"		" "	"	9.59	" = "	42		35 350
11	"		" "	200	9.55	" = "	56		33 840
12	"		" "	"	9.61	" = "	56		34 000
467	"		" "	240	9.19	" = 6"	66		29 980
3	"		" "	250	9.74	" = 8"	70		33 880
4	"		" "	"	9.81	" = "	70		33 660
5	"		" "	300	10.04	" = "	85		34 130
6	"		" "	"	10.00	" = "	85		31 930
468	"		" "	"	9.19	" = 6"	82		32 000
26	"		" "	350	9.30	" = 8"	96		32 180
27	"		" "	"	9.57	" = "	97		29 380

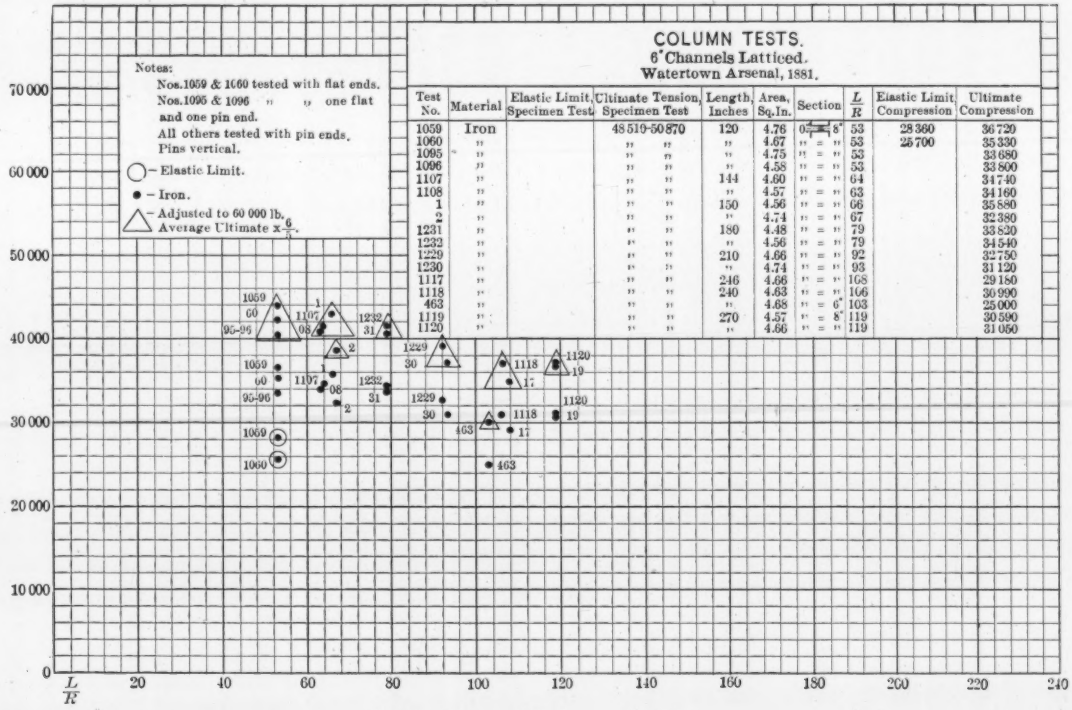
FIG. 5.



COLUMN TESTS
8″ Channels Latticed.
Watertown Arsenal, 1881.

Test No.	Material	Elastic Limit, Specimen Test.	Ultimate Tension, Specimen Test.	Length, Inches	Area, Sq. In.	Section.	$\frac{L}{R}$	Elastic Limit, Compression.	Ultimate Compression.
1111	Iron,		48 510-50 870	160	7.52	$\alpha\frac{1}{2}$ 8″	53		34 810
1112	,,		,, ,,	,,	7.50	,, = ,,	53		35 240
1113	,,		,, ,,	200	7.48	,, = ,,	67		33 970
1114	,,		,, ,,	,,	7.48	,, = ,,	67		33 610
1115	,,		,, ,,	240	7.55	,, = ,,	80		32 610
1116	,,		,, ,,	,,	7.51	,, = ,,	79		32 140
465	,,		,, ,,	,,	7.75	,, = 6″	80		27 750
1123	,,		,, ,,	280	7.99	,, = 8″	95		32 230
1124	,,		,, ,,	,,	7.67	,, = ,,	94		31 370
466	,,		,, ,,	300	7.75	,, = 6″	100		26 000
24	,,		,, ,,	320	7.78	,, = 8″	109		31 350
25	,,		,, ,,	,,	7.75	,, = ,,	108		27 850
22	,,		,, ,,	360	7.81	,, = ,,	122		24 850
23	,,		,, ,,	,,	7.80	,, = ,,	122		26 920

FIG. 6.



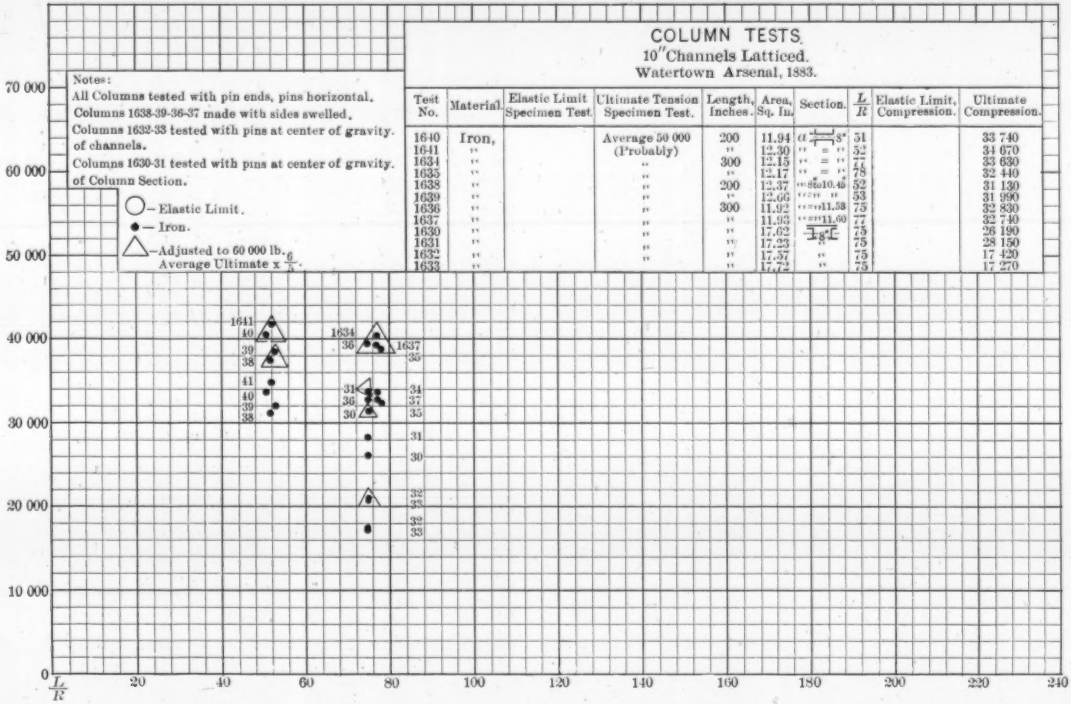
COLUMN TESTS.

6" Channels Latticed.

Watertown Arsenal, 1881.

Test No.	Material	Elastic Limit, Specimen Test	Ultimate Tension, Specimen Test	Length, Inches	Area, Sq. In.	Section	L/R	Elastic Limit Compression	Ultimate Compression
1059	Iron		48 510-50 870	120	4.76	0 [6"	53	28 360	36 720
1060	"		" "	"	4.67	" = "	53	25 700	35 330
1095	"		" "	"	4.75	" = "	53		33 680
1096	"		" "	"	4.58	" = "	53		33 800
1107	"		" "	144	4.60	" = "	64		34 740
1108	"		" "	"	4.57	" = "	63		34 160
1	"		" "	150	4.56	" = "	66		35 880
2	"		" "	"	4.74	" = "	67		32 380
1231	"		" "	180	4.48	" = "	79		33 820
1232	"		" "	"	4.56	" = "	79		34 540
1229	"		" "	210	4.66	" = "	92		32 750
1230	"		" "	"	4.74	" = "	93		31 120
1117	"		" "	246	4.66	" = "	168		29 180
1118	"		" "	240	4.63	" = "	166		30 990
463	"		" "	"	4.68	" = 6"	103		25 000
1119	"		" "	270	4.57	" = 8"	119		30 590
1120	"		" "	"	4.66	" = "	119		31 050

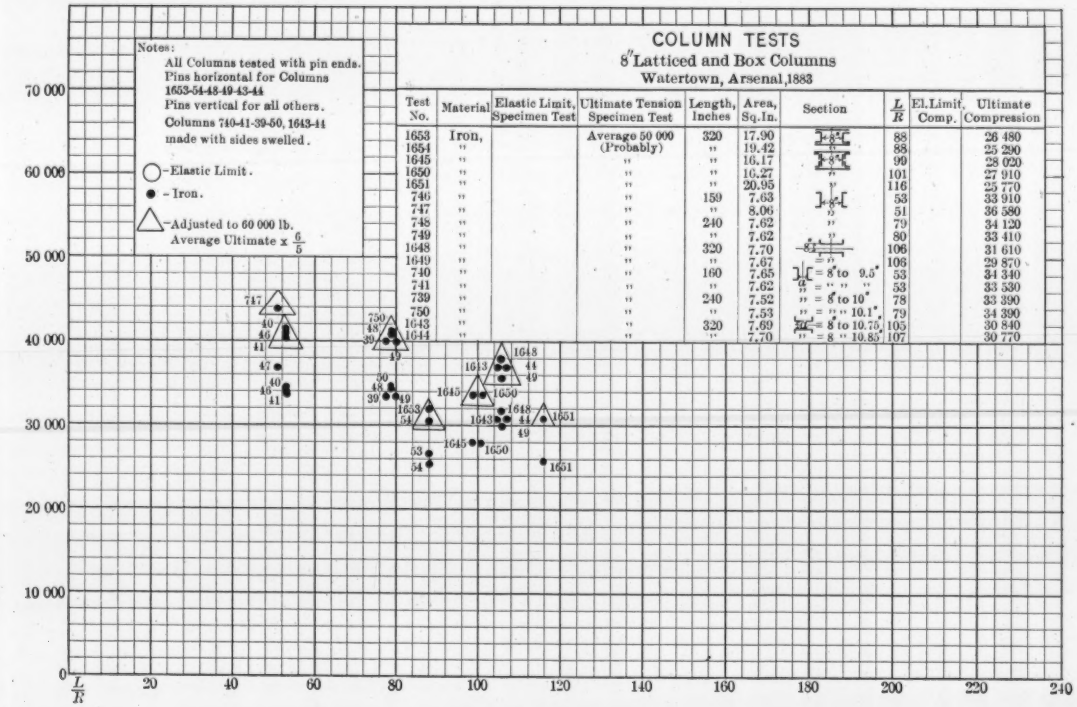
FIG. 7.



COLUMN TESTS.
10″ Channels Latticed.
Watertown Arsenal, 1883.

Test No.	Material.	Elastic Limit Specimen Test.	Ultimate Tension Specimen Test.	Length, Inches.	Area, Sq. In.	Section.	$\frac{L}{R}$	Elastic Limit, Compression.	Ultimate Compression.
1640	Iron,		Average 50 000	200	11.94	α 8″	51		33 740
1641	"		(Probably)	"	12.30	" = "	52		34 670
1634	"		"	300	12.15	" = "	77		33 630
1635	"		"	"	12.17	" = "	78		32 440
1638	"		"	200	12.37	" = 8″ to 10.45″	52		31 130
1639	"		"	"	12.66	" = " "	53		31 990
1636	"		"	300	11.92	" = " 11.53	75		32 830
1637	"		"	"	11.93	" = " 11.60	77		32 740
1630	"		"	"	17.62	8″	75		26 190
1631	"		"	"	17.23	"	75		28 150
1632	"		"	"	17.57	"	75		17 420
1633	"		"	"	17.72	"	75		17 270

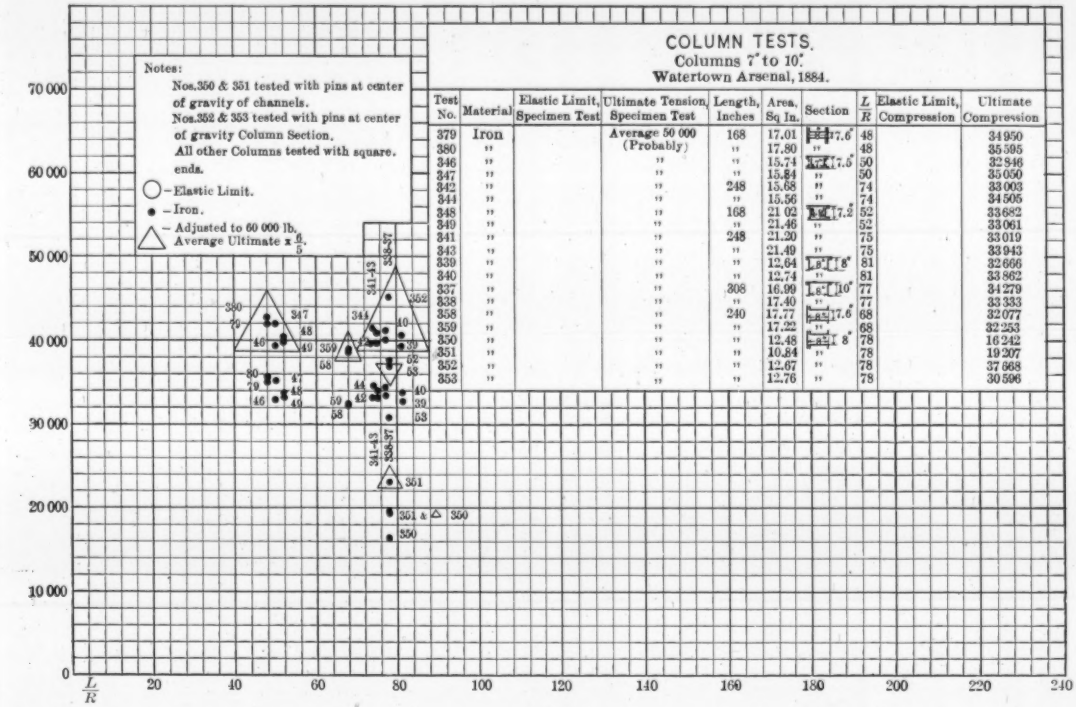
FIG. 8.



COLUMN TESTS
8" Latticed and Box Columns
Watertown, Arsenal, 1883

Test No.	Material	Elastic Limit, Specimen Test	Ultimate Tension Specimen Test	Length, Inches	Area, Sq. In.	Section	L/R	El. Limit, Comp.	Ultimate Compression
1653	Iron,		Average 50 000 (Probably)	320	17.90		88		26 480
1654	"		"	"	19.42		88		25 290
1645	"		"	"	16.17		99		28 020
1650	"		"	"	16.27		101		27 910
1651	"		"	"	20.95		116		25 770
746	"		"	159	7.63		53		33 910
747	"		"	"	8.06		51		36 580
748	"		"	240	7.62		79		34 120
749	"		"	"	7.62		80		33 410
1648	"		"	320	7.70		106		31 610
1649	"		"	"	7.67		106		29 870
740	"		"	160	7.65		53		34 340
741	"		"	"	7.62		53		33 530
739	"		"	240	7.52		78		33 390
750	"		"	"	7.53		79		34 390
1643	"		"	320	7.69		105		30 840
1644	"		"	"	7.70		107		30 770

FIG. 9.

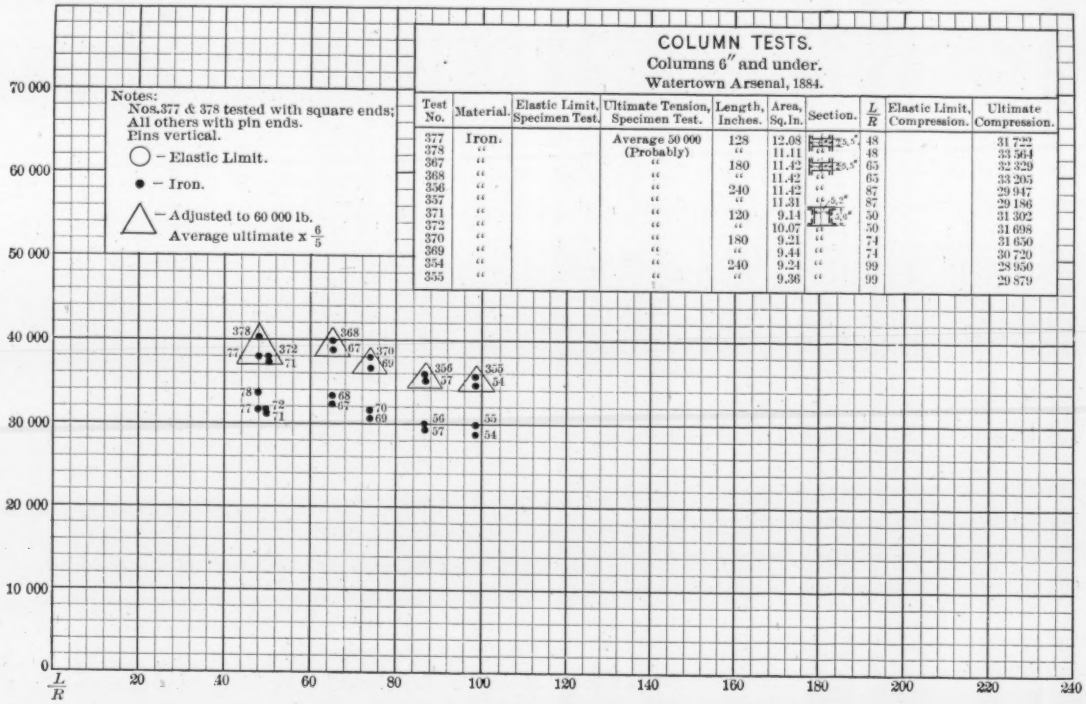


COLUMN TESTS.

Columns 7" to 10".
Watertown Arsenal, 1884.

Test No.	Material	Elastic Limit, Specimen Test	Ultimate Tension, Specimen Test	Length, Inches	Area, Sq. In.	Section	L/R	Elastic Limit, Compression	Ultimate Compression
379	Iron		Average 50 000 (Probably)	168	17.01	[7.6"	48		34950
380	"		"	"	17.80	"	48		35595
346	"		"	"	15.74	[7.5"	50		32846
347	"		"	"	15.84	"	50		35050
342	"		"	248	15.68	"	74		33003
344	"		"	"	15.56	"	74		34505
348	"		"	168	21.02	[7.2"	52		33682
349	"		"	"	21.46	"	52		33061
341	"		"	248	21.20	"	75		33019
343	"		"	"	21.49	"	75		33943
339	"		"	"	12.64	[8"	81		32666
340	"		"	"	12.74	"	81		33862
337	"		"	308	16.99	[10"	77		34279
338	"		"	"	17.40	"	77		33333
358	"		"	240	17.77	[7.6"	68		32077
359	"		"	"	17.22	"	68		32253
350	"		"	"	12.48	[8"	78		16242
351	"		"	"	10.84	"	78		19207
352	"		"	"	12.67	"	78		37668
353	"		"	"	12.76	"	78		30596

FIG. 10.



COLUMN TESTS.

Columns 6″ and under.

Watertown Arsenal, 1884.

Test No.	Material.	Elastic Limit, Specimen Test.	Ultimate Tension, Specimen Test.	Length, Inches.	Area, Sq. In.	Section.	$\frac{L}{R}$	Elastic Limit, Compression.	Ultimate Compression.
377	Iron.		Average 50 000 (Probably)	128	12.08		48		31 722
378	"		"	"	11.11		48		33 564
367	"		"	180	11.42		65		32 329
368	"		"	"	11.42		65		33 205
356	"		"	240	11.42	"	87		29 947
357	"		"	"	11.31		87		29 186
371	"		"	120	9.14		50		31 302
372	"		"	"	10.07	"	50		31 698
370	"		"	180	9.21	"	74		31 650
369	"		"	"	9.44	"	74		30 720
354	"		"	240	9.24	"	99		28 950
355	"		"	"	9.36	"	99		29 879

FIG. 11.

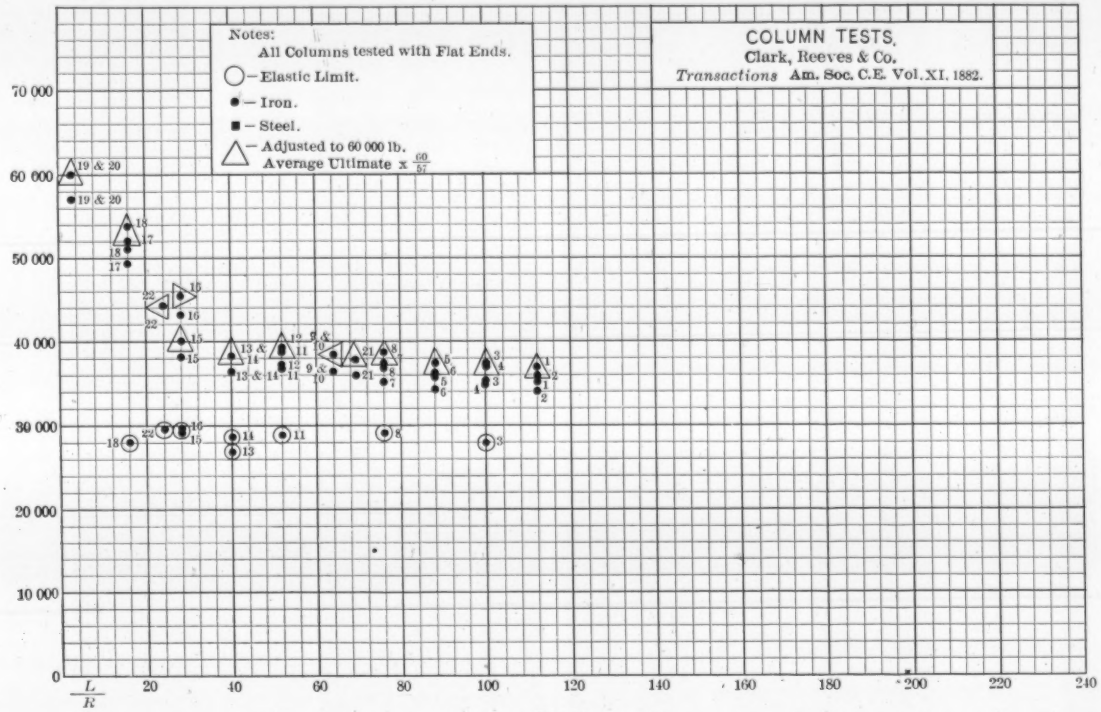

Notes:
All Columns tested with Flat Ends.
○ — Elastic Limit.
● — Iron.
■ — Steel.
△ — Adjusted to 60 000 lb.
Average Ultimate x 60/67
COLUMN TESTS.
Clark, Reeves & Co.
Transactions Am. Soc. C.E. Vol. XI. 1882.
70 000
60 000
50 000
40 000
30 000
20 000
10 000
0
L/R
20
40
60
80
100
120
140
160
180
200
220
240

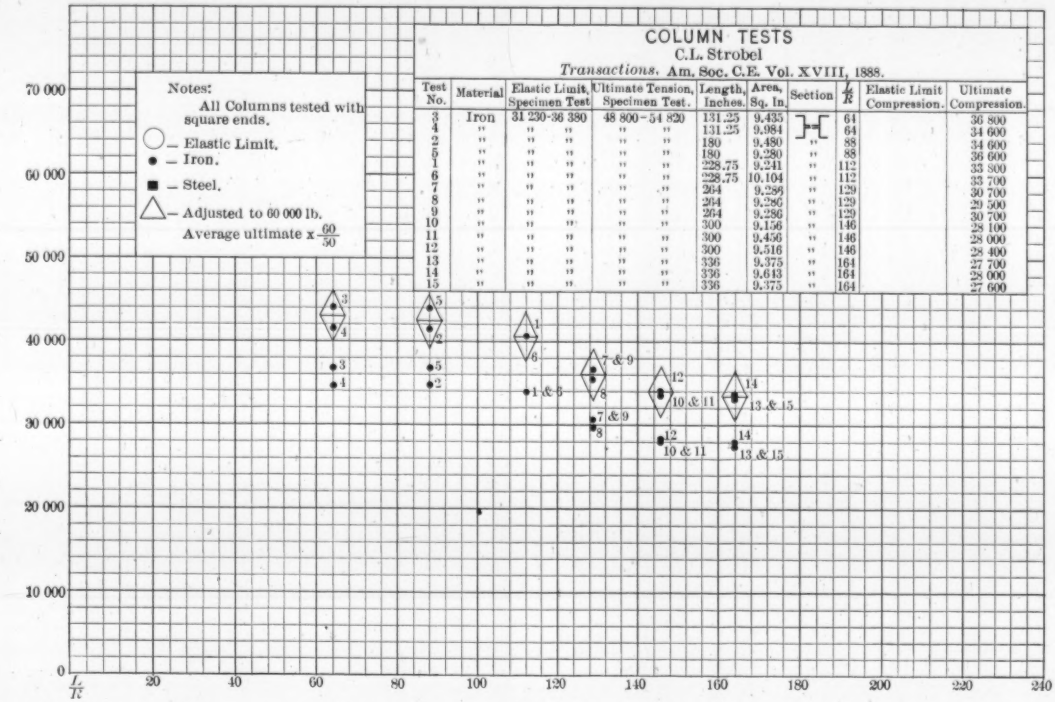
FIG. 12.

COLUMN TESTS
Clark, Reeves & Co.

Transactions, Am. Soc. C.E. Vol. XI, 1882.

Test No.	Material	Elastic Limit, Specimen Test.	Ultimate Tension, Specimen Test.	Length, Inches.	Area, Sq. In.	Section.	$\frac{l}{R}$	Elastic Limit, Compression	Ultimate Compression.
1	Iron		Clark, Reeves & Co's Specification of 1871 Calls for 55 000 to 60 000	336	12.062	⌑	112		35 150
2	"			336	12.181	"	112		34 150
3	"			300	12.233	"	100	27 960	35 270
4	"			300	12.100	"	100		35 040
5	"			264	12.371	"	88		35 570
6	"			264	12.311	"	88		34 360
7	"			228	12.023	"	76		35 365
8	"			228	12.087	"	76	29 290	36 900
9	"			192	12.000	"	64		36 580
10	"			192	12.000	"	64		36 580
11	"			156	12.185	"	52	28 800	36 857
12	"			156	12.059	"	52		37 200
13	"			120	12.248	"	40	26 940	36 480
14	"			120	12.339	"	40	28 360	36 397
15	"			84	12.265	"	28	29 350	38 157
16	"			84	11.962	"	28	29 590	43 300
17	"			48	12.081	"	16		49 500
18	"			48	12.119	"	16	28 050	51 240
19	"			8	11.903	"	2.7		57 130
20	"			8	11.903	"	2.7		57 300
21	"			302.65	18.300	☆	68.8		36 010
22	"			105.90	18.300	"	24	29 510	42 180

FIG. 13.



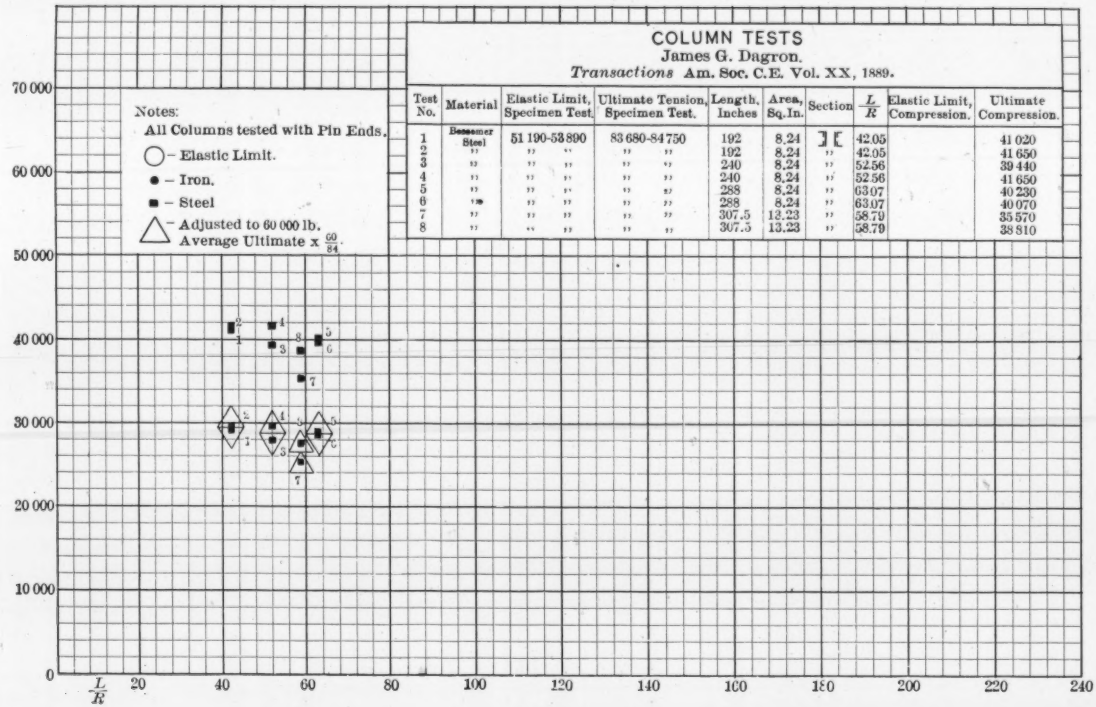
COLUMN TESTS

C.L. Strobel

Transactions, Am. Soc. C.E. Vol. XVIII, 1888.

Test No.	Material	Elastic Limit, Specimen Test	Ultimate Tension, Specimen Test.	Length, Inches.	Area, Sq. In.	Section	$\frac{L}{R}$	Elastic Limit Compression.	Ultimate Compression.
3	Iron	31 230-36 380	48 800-54 820	131.25	9.435	⊢⊣	64		36 800
4	"	" "	" "	131.25	9.584	"	64		34 600
2	"	" "	" "	180	9.480	"	88		34 600
5	"	" "	" "	180	9.280	"	88		36 600
1	"	" "	" "	228.75	9.241	"	112		33 800
6	"	" "	" "	228.75	10.104	"	112		33 700
7	"	" "	" "	264	9.286	"	129		30 700
8	"	" "	" "	264	9.296	"	129		29 500
9	"	" "	" "	264	9.286	"	129		30 700
10	"	" "	" "	300	9.156	"	146		28 100
11	"	" "	" "	300	9.456	"	146		28 000
12	"	" "	" "	300	9.516	"	146		28 400
13	"	" "	" "	336	9.375	"	164		27 700
14	"	" "	" "	336	9.643	"	164		28 000
15	"	" "	" "	336	9.375	"	164		27 600

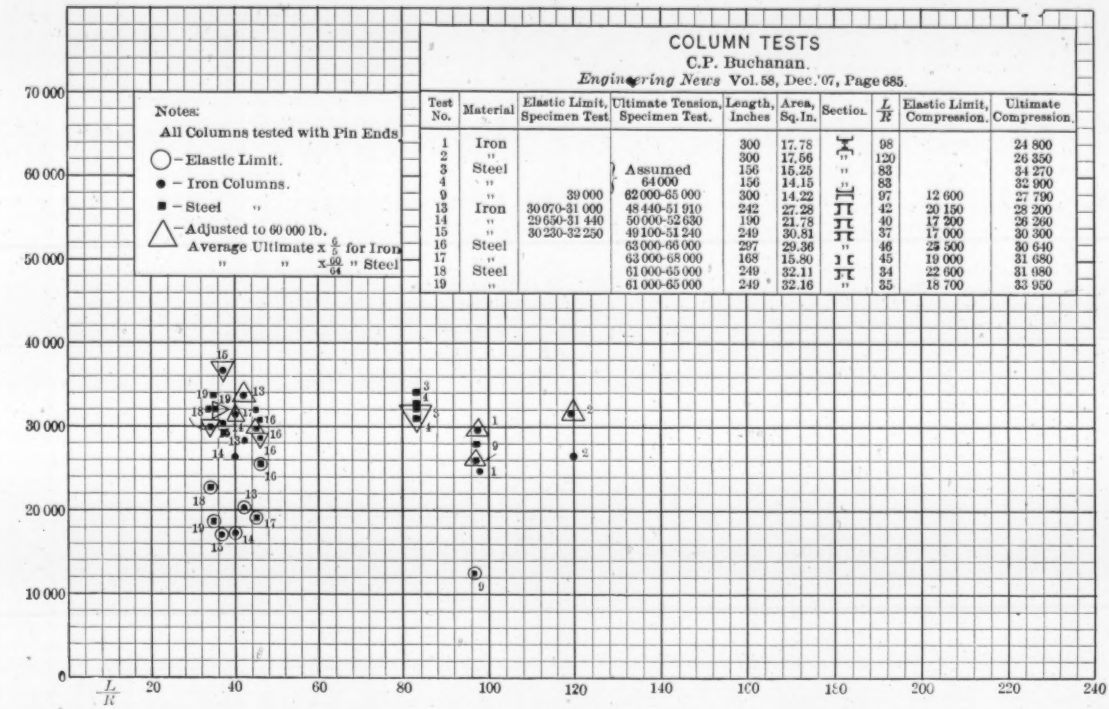
FIG. 14.



COLUMN TESTS
James G. Dagron.
Transactions Am. Soc. C.E. Vol. XX, 1889.

Test No.	Material	Elastic Limit, Specimen Test.	Ultimate Tension, Specimen Test.	Length, Inches	Area, Sq. In.	Section	$\frac{L}{R}$	Elastic Limit, Compression.	Ultimate Compression.
1	Bessemer Steel	51 190-53 890	83 680-84 750	192	8.24	][	42.05		41 020
2	"	" "	" "	192	8.24	"	42.05		41 650
3	"	" "	" "	240	8.24	"	52.56		39 440
4	"	" "	" "	240	8.24	"	52.56		41 650
5	"	" "	" "	288	8.24	"	63.07		40 230
6	"	" "	" "	288	8.24	"	63.07		40 070
7	"	" "	" "	307.5	12.23	"	58.79		35 570
8	"	" "	" "	307.5	13.23	"	58.79		38 810

FIG. 15



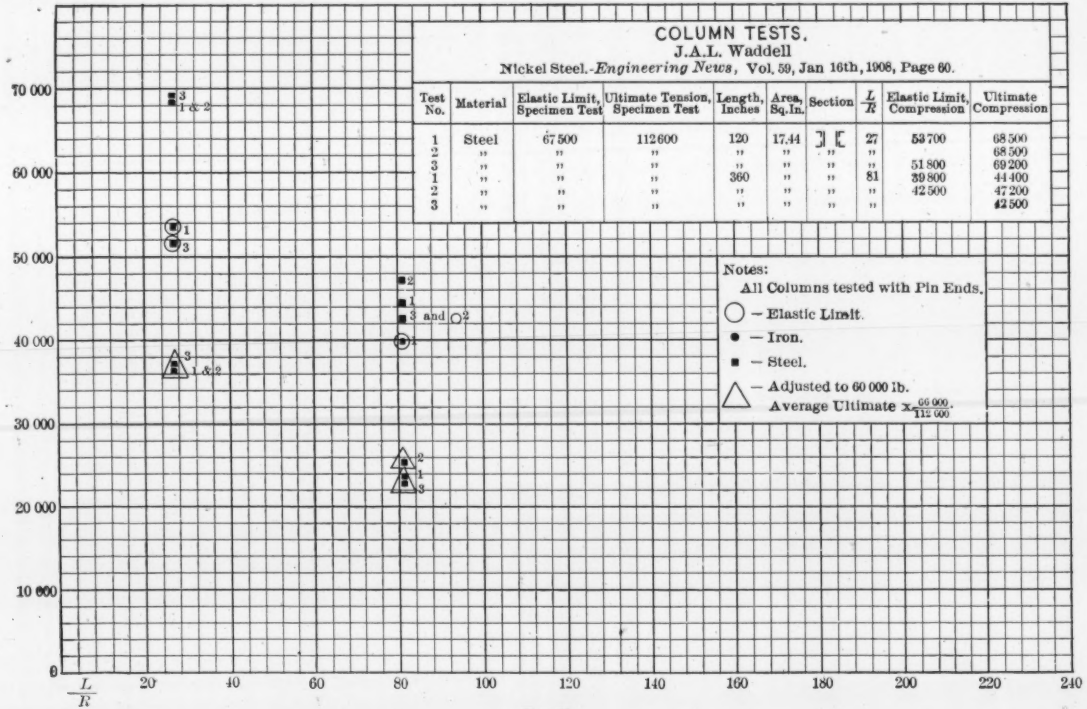
COLUMN TESTS

C.P. Buchanan.

Engineering News Vol. 58, Dec. '07, Page 685.

Test No.	Material	Elastic Limit, Specimen Test.	Ultimate Tension, Specimen Test.	Length, Inches	Area, Sq. In.	Section.	L/R	Elastic Limit, Compression.	Ultimate Compression.
1	Iron			300	17.78	H	98		24 800
2	"			300	17.56	"	120		26 350
3	Steel		Assumed	156	15.25	"	83		34 270
4	"		64 000	156	14.15	"	83		32 900
9	"	39 000	62 000-65 000	300	14.22	⊏⊏	97	12 600	27 790
13	Iron	30 070-31 000	48 440-51 910	242	27.28	⊏⊏⊏⊏	42	20 150	28 200
14	"	29 650-31 440	50 000-52 630	190	21.78	"	40	17 200	26 260
15	"	30 230-32 250	49 100-51 240	249	30.81	"	37	17 000	30 300
16	Steel		63 000-66 000	297	29.36	"	46	25 500	30 640
17	"		63 000-68 000	168	15.80	⊐⊏	45	19 000	31 680
18	Steel		61 000-65 000	249	32.11	⊐⊏	34	22 600	31 980
19	"		61 000-65 000	249	32.16	"	35	18 700	33 950

FIG. 16.



COLUMN TESTS.
J.A.L. Waddell
Nickel Steel.-*Engineering News*, Vol. 59, Jan 16th, 1908, Page 60.

Test No.	Material	Elastic Limit, Specimen Test	Ultimate Tension, Specimen Test	Length, Inches	Area, Sq.In.	Section	$\frac{L}{R}$	Elastic Limit, Compression	Ultimate Compression
1	Steel	67 500	112 600	120	17.44	] [	27	53 700	68 500
2	,,	,,	,,	,,	,,	,,	,,		68 500
3	,,	,,	,,	,,	,,	,,	,,	51 800	69 200
1	,,	,,	,,	360	,,	,,	81	39 800	44 400
2	,,	,,	,,	,,	,,	,,	,,	42 500	47 200
3	,,	,,	,,	,,	,,	,,	,,		42 500

Notes:
All Columns tested with Pin Ends.
○ — Elastic Limit.
● — Iron.
■ — Steed.
△ — Adjusted to 60 000 lb.
Average Ultimate $\times \frac{60\,000}{112\,600}$.

FIG. 17.

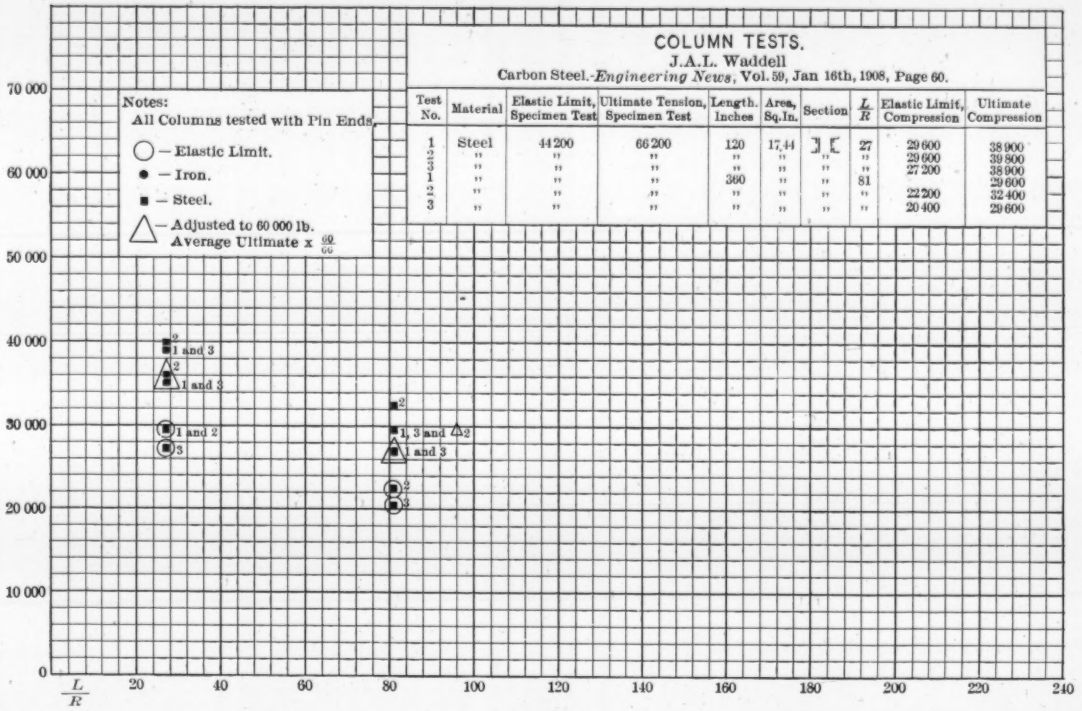

COLUMN TESTS.
J.A.L. Waddell
Carbon Steel.-Engineering News, Vol. 59, Jan 16th, 1908, Page 60.
Test No. | Material | Elastic Limit, Specimen Test | Ultimate Tension, Specimen Test | Length. Inches | Area, Sq.In. | Section | L/R | Elastic Limit, Compression | Ultimate Compression
1 | Steel | 44200 | 66200 | 120 | 17.44 |] " [| 27 | 29600 | 38900
2 | " | " | " | " | " | " | " | 29600 | 39800
3 | " | " | " | " | " | " | " | 27200 | 38900
1 | " | " | " | 360 | " | " | 81 | | 29600
2 | " | " | " | " | " | " | " | 22200 | 32400
3 | " | " | " | " | " | " | " | 20400 | 29600
Notes:
All Columns tested with Pin Ends.
○ — Elastic Limit.
● — Iron.
■ — Steel.
△ — Adjusted to 60 000 lb. Average Ultimate x 69/66
70 000
60 000
50 000
40 000
30 000
20 000
10 000
0
L/R
20
40
60
80
100
120
140
160
180
200
220
240
2 1 and 3
2 1 and 3
1 and 2 3
2 1, 3 and 2 1 and 3
2 3

FIG. 18.

COLUMN TESTS.

Lap-Welded Steel Tubing.

Watertown Arsenal, "Tests of Metals," 1908.

Test No.	Material	Elastic Limit, Specimen Test	Ultimate Tension, Specimen Test	Length, Inches	Area, Sq.In.	Section	$\frac{L}{R}$	Elastic Limit, Compression	Ultimate Compression
1895	Steel	35 000-38 610	56 390-60 830	20.56	5.15	○	12.5	32 000	67 000
6	"	" "	" "	"	5.24	"	12.5	33 000	66 350
7	"	" "	" "	"	5.24	"	12.5	33 000	66 180
1891	"	" "	" "	41.00	5.34	"	25.0		41 700
2	"	" "	" "	"	5.25	"	"	30 000	40 610
3	"	" "	" "	"	5.17	"	"	29 000	39 850
1898	"	" "	" "	41.12	5.10	"	"	31 000	46 000
9	"	" "	" "	"	5.18	"	"	31 000	43 260
1900	"	" "	" "	"	5.36	"	"	29 000	41 420
1	"	" "	" "	61.69	5.06	"	37.5	32 000	34 000
2	"	" "	" "	"	5.23	"	"	30 000	32 000
3	"	" "	" "	"	5.34	"	"	27 000	31 740
1888	"	" "	" "	82.25	5.24	"	50		28 820
9	"	" "	" "	"	5.19	"	"		32 000
1890	"	" "	" "	"	5.21	"	"		32 000
1904	"	" "	" "	"	5.33	"	"	28 000	30 000
5	"	" "	" "	"	5.06	"	"	31 000	32 000
6	"	" "	" "	"	5.25	"	"	30 000	32 000
7	"	" "	" "	102.31	5.20	"	62.5	32 000	35 630
8	"	" "	" "	"	5.08	"	"		31 000
9	"	" "	" "	"	5.26	"	"	29 000	30 000
1894	"	" "	" "	120.50	5.16	"	73.3		33 000
1885	"	" "	" "	123.50	5.23	"	75		33 000
6	"	" "	" "	"	5.19	"	"		33 000
7	"	" "	" "	"	5.20	"	"		31 000
1882	"	" "	" "	164.50	5.26	"	100		31 000
3	"	" "	" "	"	5.21	"	"		28 000
4	"	" "	" "	"	5.17	"	"		31 000
1879	"	" "	" "	205.50	5.21	"	125		29 000
80	"	" "	" "	"	5.07	"	"		26 030
1	"	" "	" "	"	5.21	"	"		29 000
1876	"	" "	" "	246.75	5.11	"	150		26 000
7	"	" "	" "	"	5.17	"	"		27 000
8	"	" "	" "	"	5.16	"	"		23 000
1873	"	" "	" "	288.00	5.04	"	175		25 000
4	"	" "	" "	"	5.06	"	"		24 000
5	"	" "	" "	"	5.20	"	"		27 000

Average: 36 250

Average: 58 310

FIG. 19.

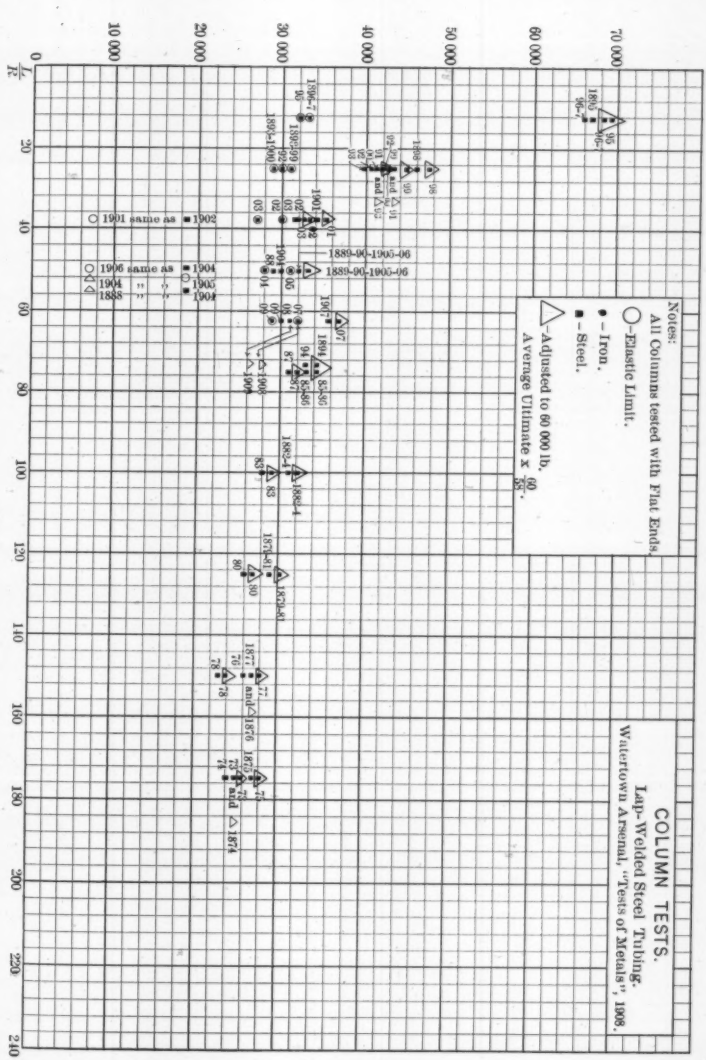
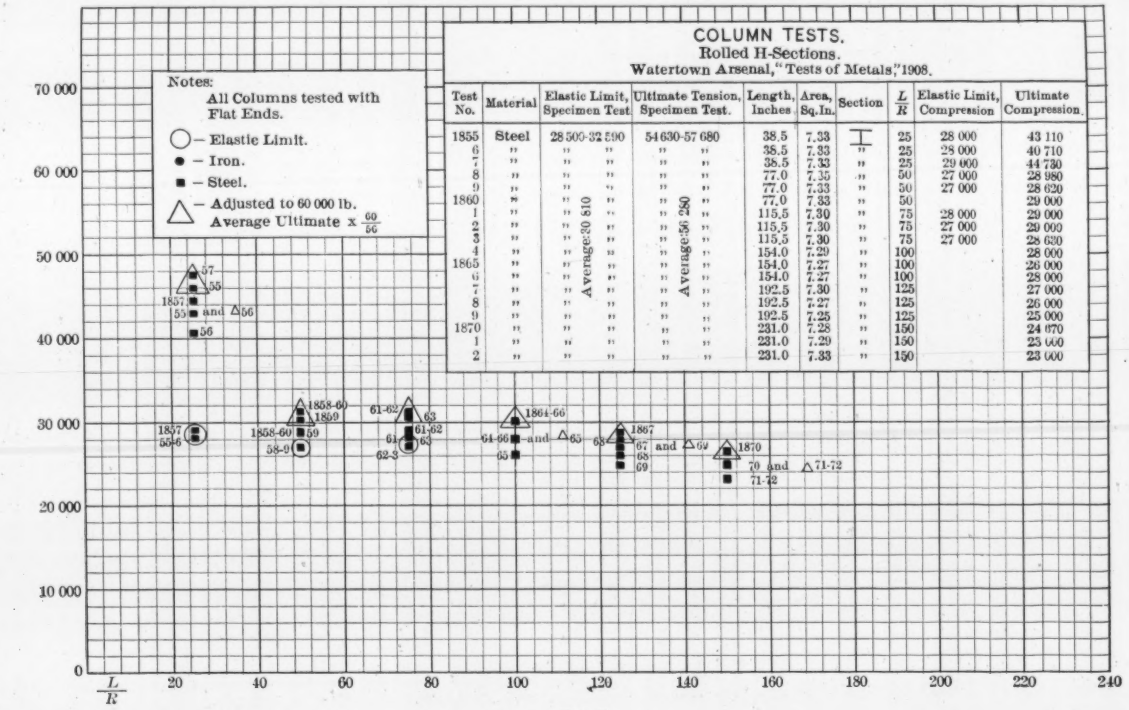

COLUMN TESTS.
Lap-Welded Steel Tubing.
Watertown Arsenal, "Tests of Metals", 1908.
Notes:
All Columns tested with Flat Ends.
○–Elastic Limit.
●–Iron.
■–Steel.
△–Adjusted to 60 000 lb. Average Ultimate x 60/55.
1889-90-1905-06
1889-90-1905-06
○ 1901 same as ■ 1902
1906 same as ■ 1904
1904 " ○ 1905
1888 " △ 1901
0
20
40
60
80
100
120
140
160
180
200
220
240
10 000
20 000
30 000
40 000
50 000
60 000
70 000
l/r

Fig. 20.

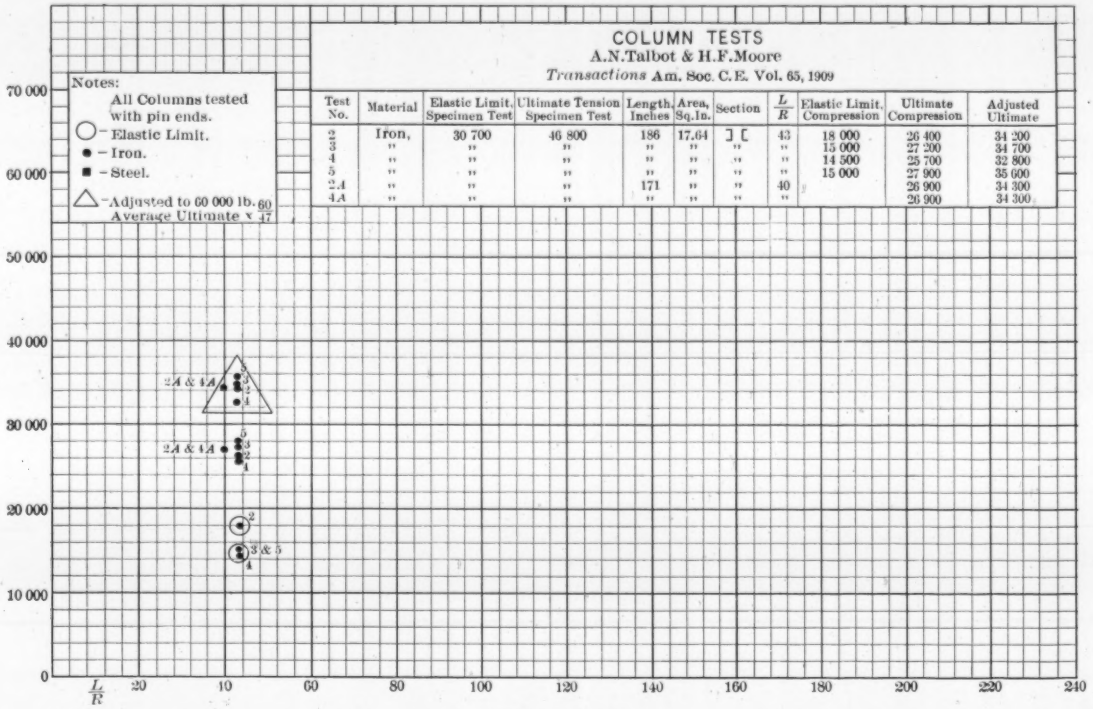


COLUMN TESTS.
Rolled H-Sections.
Watertown Arsenal, "Tests of Metals," 1908.

Test No.	Material	Elastic Limit, Specimen Test.	Ultimate Tension, Specimen Test.	Length, Inches.	Area, Sq. In.	Section	$\frac{L}{R}$	Elastic Limit, Compression.	Ultimate Compression.
1855	Steel	28 500-32 590	54 630-57 680	38.5	7.33	I	25	28 000	43 110
6	"	" "	" "	38.5	7.33	"	25	28 000	40 710
7	"	" "	" "	38.5	7.33	"	25	29 000	44 730
8	"	" "	" "	77.0	7.35	"	50	27 000	28 980
9	"	" "	" "	77.0	7.33	"	50	27 000	28 620
1860	"	" "	" "	77.0	7.33	"	50		29 000
1	"	" "	" "	115.5	7.30	"	75	28 000	29 000
2	"	" "	" "	115.5	7.30	"	75	27 000	29 000
3	"	" "	" "	115.5	7.30	"	75	27 000	28 620
4	"	" "	" "	154.0	7.29	"	100		28 000
1865	"	" "	" "	154.0	7.27	"	100		26 000
6	"	" "	" "	154.0	7.27	"	100		28 000
7	"	" "	" "	192.5	7.30	"	125		27 000
8	"	" "	" "	192.5	7.27	"	125		26 000
9	"	" "	" "	192.5	7.25	"	125		25 000
1870	"	" "	" "	231.0	7.28	"	150		24 670
1	"	" "	" "	231.0	7.29	"	150		25 000
2	"	" "	" "	231.0	7.33	"	150		23 000

Average 30 810 | Average 56 250

FIG. 21.



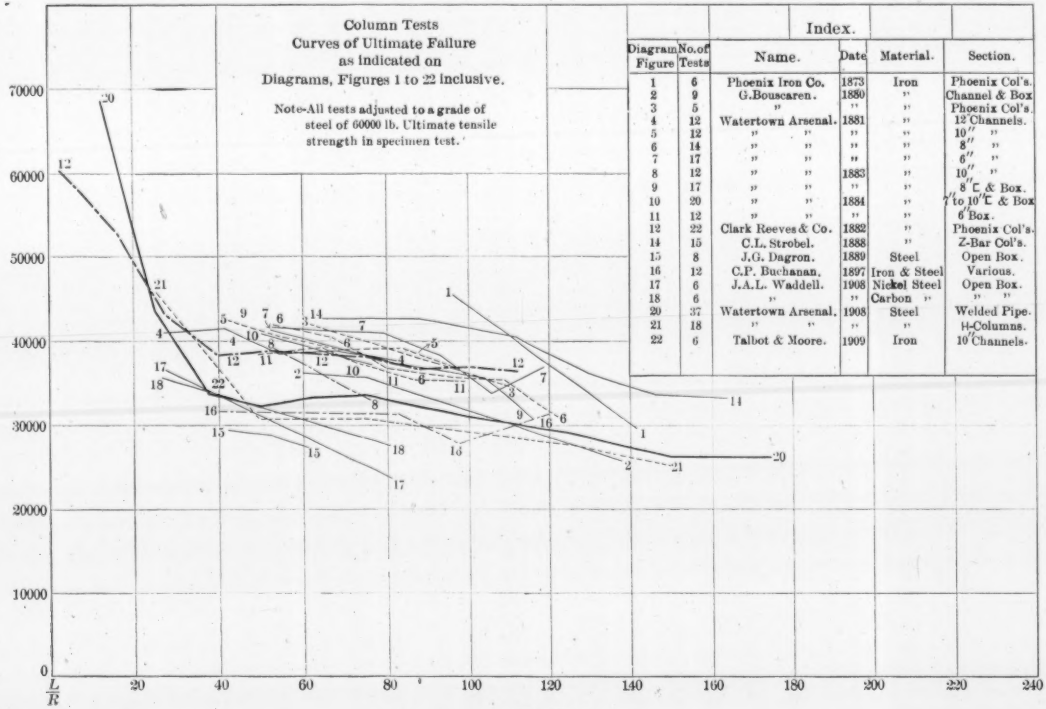
COLUMN TESTS

A.N.Talbot & H.F.Moore

Transactions Am. Soc. C.E. Vol. 65, 1909

Test No.	Material	Elastic Limit, Specimen Test	Ultimate Tension Specimen Test	Length, Inches	Area, Sq. In.	Section	$\frac{L}{R}$	Elastic Limit, Compression	Ultimate Compression	Adjusted Ultimate
2	Iron,	30 700	46 800	186	17.64	][	43	18 000	26 400	34 200
5	"	"	"	"	"	"	"	15 000	27 200	34 700
4	"	"	"	"	"	"	"	14 500	25 700	32 800
5	"	"	"	"	"	"	"	15 000	27 900	35 600
2A	"	"	"	171	"	"	40		26 900	34 300
4A	"	"	"	"	"	"	"		26 900	34 300

FIG. 22.



Index.

Diagram Figure	No. of Tests	Name.	Date	Material.	Section.
1	6	Phoenix Iron Co.	1873	Iron	Phoenix Col's.
2	9	G.Bouscaren.	1880	"	Channel & Box
3	5	"	"	"	Phoenix Col's.
4	12	Watertown Arsenal.	1881	"	12" Channels.
5	12	" "	"	"	10" "
6	14	" "	"	"	8" "
7	17	" "	"	"	6" "
8	12	" "	1883	"	10" "
9	17	" "	"	"	8" ⊏ & Box.
10	20	" "	1884	"	7" to 10" ⊏ & Box
11	12	" "	"	"	6" Box.
12	22	Clark Reeves & Co.	1882	"	Phoenix Col's.
14	15	C.L. Strobel.	1888	"	Z-Bar Col's.
15	8	J.G. Dagron.	1889	Steel	Open Box.
16	12	C.P. Buchanan.	1897	Iron & Steel	Various.
17	6	J.A.L. Waddell.	1908	Nickel Steel	Open Box.
18	6	"	"	Carbon "	" "
20	37	Watertown Arsenal.	1908	Steel	Welded Pipe.
21	18	" "	"	"	H-Columns.
22	6	Talbot & Moore.	1909	Iron	10" Channels.

FIG. 23.

DISCUSSION

Mr. Johnson.

THOMAS H. JOHNSON, M. AM. SOC. C. E. (by letter).—In this Progress Report the Committee has brought together many column tests recorded heretofore, and has sought to prepare the way for further study and analysis by adjusting the results to a uniform grade of metal, for which it has chosen that grade which has an ultimate tensile strength of 60 000 lb. per sq. in. For this purpose it assumes that the ultimate strengths of different grades of metal in tension and compression are proportional.

It is not the intention of the writer to criticize the latter assumption, but to offer objection to the manner of its application.

The Committee has apparently lost sight of the fact that the stresses in a column are a combination of compression and bending, and that, as the length ratio $\left(\frac{l}{r}\right)$ increases, the bending stresses become more and more dominant, until at last the compression disappears, and the column passes under the law represented by Euler's equation, $\frac{P}{A} = \frac{E\,a}{\left(\frac{l}{r}\right)^2}$.

The only physical properties embraced in this equation are, the factor, a, representing the form of end bearing, and the modulus of elasticity, E, a factor which is practically constant for all grades of iron and steel. The variations in this factor, as found for different grades of steel, are not greater than are frequently found for different specimens of the same steel.

This fact being recognized, it follows that, within the field of the higher length ratios, where Euler's equation applies, all grades of steel, *ceteris paribus*, give similar results; and in the field of the shorter length ratios the different grades of metal give results converging with increasing length ratios.

Now, the Committee has applied the ratio, found as stated, to all tests in the respective groups, regardless of the value of $\frac{l}{r}$ in each instance; but that ratio is applicable only at the axis of abscissas $\left(\frac{l}{r} = 0\right)$, and the adjustment should diminish to 0 at that value where Euler's equation begins to apply. Hence, the method adopted by the Committee will be misleading and deceptive by applying unduly large adjustments to tests with the larger values of $\frac{l}{r}$.

The value of $\frac{l}{r} = 0$ will vary with the form of end bearing, and the grade of material, and is indeterminate at this stage of the Com-

Mr. Johnson

mittee's work, unless it assumes to be correct, either the equation heretofore proposed by the writer, or that of the late J. B. Johnson, M. Am. Soc. C. E., or perhaps some other; but, if it is the intention that the work of this Committee shall be wholly independent of previous investigations and studies, such assumption is scarcely in order.

But the fact that the correction for adjustment must diminish to 0 at some definite value of $\frac{l}{r}$ in each group of tests cannot be avoided, and any conclusions based on the uniformly applied ratios of adjustment will be vitiated thereby.

It seems best to call attention to this point now, before the work of the Committee has progressed to the point of definite conclusions.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1141

PROGRESS REPORT OF SPECIAL COMMITTEE ON BITUMINOUS MATERIALS FOR ROAD CONSTRUCTION.*

Your Special Committee, appointed May 4th, 1909, to report on "Bituminous Materials for Road Construction and on Standards for Their Test and Use," respectfully presents the following Progress Report.

On receiving notice of its appointment, your Committee organized by electing W. W. Crosby, Chairman, and A. H. Blanchard, Secretary. Your Committee agreed at once that the available information concerning its subject was so diversified in both form and character as to render it impossible of satisfactory use by the Committee, and the question of securing additional information on uniform lines was considered.

As actual construction had already begun in many instances, it seemed imperative to establish at the earliest possible moment a schedule of information concerning such work, which should cover all the points on which it appeared that information was necessary, and yet which should, if possible, not be so extensive in form as to deter anyone from attempting to respond to a large portion of the questions asked.

In preparing this schedule, the earlier reports from many sources were considered, and, as far as seemed to the Committee practicable, an effort was made to include in its schedule the points that seemed to others important. It was also found necessary, in order to secure uniformity, to suggest methods for arriving at some of the figures requested. As far as it seemed practicable, already standardized methods were adopted, and only such changes in ordinary methods were

* Presented to the Annual Meeting, January 19th, 1910.

inserted as seemed necessary for accuracy or improvement. In some instances new methods had to be devised.

The Committee submitted a list of analyses and methods of testing bituminous materials to many engineers and chemists, conversant with the subject, for criticism and suggestions. After considerable correspondence, and a series of meetings of the Committee, and after continued revision, the final list was adopted as representing most nearly the views of the majority.

A circular letter was then prepared, explaining the intentions of the Committee and inviting the co-operation and assistance of those interested in its work.

The letter and forms were printed and distributed about August 1st to National, State, County, Town and City Highway Engineers and Officials, and to Chemists, Manufacturers of, and Agents for, Bituminous Materials. More than 800 letters were sent out during August and September.

From replies already received, your Committee feels greatly encouraged by the interest manifested in its work, in spite of, as we recognize, the amount of work requested by it of its correspondents. The States of Maine, Massachusetts, Maryland, Pennsylvania, New Hampshire, New York, and Rhode Island have agreed to report to the Committee on the bituminous macadam roads constructed during 1909, including the analyses of the bituminous materials used, made in accordance with the methods suggested by the Committee.

At this date, your Committee can scarcely report more than "Progress." Conclusions cannot be drawn on most points until the results of the action of the varying seasons and traffic conditions shall have become apparent. It is hoped that partial conclusions may be justified in time to report at the next Annual Meeting.

The Committee begs leave to express here its profound appreciation of the courtesy and material assistance rendered it in its work by your Secretary, Mr. Charles Warren Hunt, and also by the engineers, chemists, and manufacturers with whom it has had the good fortune to come in contact, and to sign itself

Very respectfully,

W. W. CROSBY, *Chairman.*
H. K. BISHOP.
A. W. DEAN.
A. H. BLANCHARD, *Secretary.*

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1142

PROGRESS REPORT OF SPECIAL COMMITTEE ON CONCRETE AND REINFORCED CONCRETE.*

WITH DISCUSSION BY MESSRS. WILBUR J. WATSON, JOHN STEPHEN SEWELL, CLARENCE W. NOBLE, S. BENT RUSSELL, J. R. WORCESTER, W. J. DOUGLAS, AND C. A. P. TURNER.

MAJORITY REPORT.†

I. INTRODUCTION.

Special Committees were appointed by the American Society of Civil Engineers, American Society for Testing Materials, American Railway Engineering and Maintenance of Way Association, and the Association of American Portland Cement Manufacturers, for the purpose of investigating current practice and providing definite information concerning the properties of concrete and reinforced concrete and to recommend necessary factors and formulas required in the design of structures in which these materials are used.

At the Annual Convention of the American Society of Civil Engineers held at Asheville, N. C., June 11th, 1903, the following resolution was adopted:

It is the sense of this meeting that a Special Committee be appointed to take up the question of concrete and steel-concrete, and that such committee co-operate with the American Society for Testing Materials, and the American Railway Engineering and Maintenance of Way Association.

Following the adoption of this resolution, a Special Committee on concrete and steel-concrete was appointed by the Board of Direction on May 31st, 1904. At the Annual Meeting, held January 18th, 1905,

* Presented to the Business Meeting of the Annual Convention, July 7th, 1909.

† For Minority Report, see page 465.

the title of this special committee was, at the request of the Committee, changed to "Special Committee on Concrete and Reinforced Concrete."

At the annual meeting of the American Society for Testing Materials held July 1st, 1903, at the Delaware Water Gap, the following resolution was unanimously adopted:

That the Executive Committee be requested to consider the desirability of appointing a committee on "Reinforced Concrete," with a view of co-operating with the committees of other societies in the study of the subject.

At the meeting of the Executive Committee of that Society, held December 5th, 1903, a special committee on "Reinforced Concrete" was appointed.

The American Railway Engineering and Maintenance of Way Association appointed a Committee on Masonry on July 20th, 1899, with instructions, as a part of its duties, to prepare specifications for concrete masonry. A preliminary set of specifications for Portland cement concrete was reported to and adopted by the Association on March 19th, 1903. At the meeting held in Chicago on March 17th, 1904, the Committee on Masonry was authorized to co-operate with the Special Committee on Concrete and Reinforced Concrete of the American Society of Civil Engineers, and following this action a special subcommittee was appointed.

At a meeting of the several special committees representing the above-mentioned societies, held at Atlantic City, N. J., June 17th, 1904, arrangements were completed for collaborating the work of these several committees through the formation of the Joint Committee on Concrete and Reinforced Concrete. Mr. C. C. Schneider was elected temporary chairman and Professor A. N. Talbot was elected temporary secretary. The proposed plan of action of the special committee of the American Society of Civil Engineers was outlined, involving the appointment of subcommittees on Plan and Scope, on Tests, and on Ways and Means.

Mr. Schneider and Mr. Schaub, as Chairman and Secretary of the Special Committee of the American Society of Civil Engineers, were elected permanent Chairman and Secretary of the Joint Committee on Concrete and Reinforced Concrete. Mr. Emil Swensson was elected Vice-Chairman, and, on the resignation of Mr. Schaub, Mr. Richard L. Humphrey was elected Secretary.

The following is the organization of the Joint Committee:

OFFICERS.

Chairman—C. C. Schneider.
Vice-Chairman—Emil Swensson.
Secretary—Richard L. Humphrey.

MEMBERS.

American Society of Civil Engineers (Special Committee on Concrete and Reinforced Concrete):

Greiner, J. E., Consulting Engineer, Baltimore and Ohio Railroad, Baltimore, Md.

Hatt, W. K., Professor of Civil Engineering, Purdue University, Lafayette, Ind.

Hoff, Olaf, Consulting Engineer, New York, N. Y.

Humphrey, Richard L., Consulting Engineer, Engineer in Charge, Structural Materials Testing Laboratories, U. S. Geological Survey, Philadelphia, Pa.

Lesley, R. W., President, American Cement Company, Philadelphia, Pa.

Schaub, J. W., Consulting Engineer, Chicago, Ill.*

Schneider, C. C., Consulting Engineer, Philadelphia, Pa.

Swensson, Emil, Consulting Engineer, Pittsburg, Pa.

Talbot, A. N., Professor of Municipal and Sanitary Engineering, Charge of Theoretical and Applied Mechanics, University of Illinois, Urbana, Ill.

Worcester, J. R., Consulting Engineer, Boston, Mass.

American Society for Testing Materials (Committee on Reinforced Concrete):

Fuller, William B., Consulting Engineer, New York, N. Y.

Heidenreich, E. Lee, Consulting Engineer, New York, N. Y.

Humphrey, Richard L., Consulting Engineer, Engineer in Charge, Structural Materials Testing Laboratories, U. S. Geological Survey, Philadelphia, Pa.

Johnson, Albert L., Consulting Engineer, St. Louis, Mo.

Lanza, Gaetano, Professor of Theoretical and Applied Mechanics, Massachusetts Institute of Technology, Boston, Mass.

Lesley, R. W., President, American Cement Company, Philadelphia, Pa.

Marburg, Edgar, Professor of Civil Engineering, University of Pennsylvania, Philadelphia, Pa.

Mills, Charles M., Principal Assistant Engineer, Philadelphia Rapid Transit Company, Philadelphia, Pa.

Moisseiff, Leon S., Assistant Engineer, Department of Bridges, New York, N. Y.

Quimby, Henry H., Assistant Engineer of Bridges, Bureau of Surveys, Philadelphia, Pa.

Taylor, W. P., Engineer in Charge of Testing Laboratory, Philadelphia, Pa.

Thompson, Sanford E., Consulting Engineer, Newton Highlands, Mass.

Turneaure, F. E., Dean of College of Mechanics and Engineering, University of Wisconsin, Madison, Wis.

Wagner, Samuel Tobias, Assistant Engineer, Philadelphia and Reading Railroad, Philadelphia, Pa.

Webster, George S., Chief Engineer, Bureau of Surveys, Philadelphia, Pa.

* Deceased since this Report was presented.

American Railway Engineering and Maintenance of Way Association (Subcommittee on Reinforced Concrete):
- Boynton, C. W., Inspecting Engineer, Universal Portland Cement Company, Chicago, Ill.
- Cunningham, A. O., Chief Engineer, Wabash Railroad, St. Louis, Mo.
- Moore, C. H., Engineer of Grade Crossings, Erie Railroad, New York, N. Y.
- Scribner, Gilbert H., Jr., Contracting Engineer, Chicago, Ill.
- Swain, George F., Professor of Civil Engineering, Massachusetts Institute of Technology, Boston, Mass.

Association of American Portland Cement Manufacturers (Committee on Concrete and Steel Concrete):
- Fraser, Norman D., President, Chicago Portland Cement Company, Chicago, Ill.
- Griffiths, R. E., Vice-President, American Cement Company, Philadelphia, Pa.
- Hagar, Edward M., President, Universal Portland Cement Company, Chicago, Ill.
- Newberry, Spencer B., Manager, Sandusky Portland Cement Company, Sandusky, Ohio.

The report, herein presented, embodies the present judgment of the Committee concerning the proper use of Concrete and Reinforced Concrete.

II. Adaptability of Concrete and Reinforced Concrete.

The adaptability of concrete and reinforced concrete for engineering structures, or parts thereof, is now so well established that it may be considered one of the recognized materials of construction. It has proved to be a satisfactory material, when properly used, for those purposes for which its qualities make it particularly suitable.

1. Proper Use.

Concrete is a material of very low tensile strength and capable of sustaining but very small tensile deformations without rupture; its value as a structural material depends chiefly upon its durability, its fire-resisting qualities, its strength in compression and its relatively low cost. Its strength increases generally with age.

Plain concrete or massive concrete is well adapted for structural forms in which the principal stresses are compressive. These include foundations, dams, retaining and other walls, piers, abutments, short columns and, in many cases, arches. In the design of massive concrete, the tensile strength of the material must generally be neglected.

By the use of metal reinforcement to resist the principal tensile stresses, concrete becomes available for general use in a great variety of structures and structural forms. This combination of concrete and metal is particularly advantageous in the beam, where both compression

and tension exist; it is also advantageous in the column, where the main stresses are compressive, but where cross-bending may exist. In structures resisting lateral forces it possesses advantages over plain concrete in that it may be so designed as to utilize more fully the strength rather than the weight of the material.

2. Improper Use.

Failures of reinforced concrete structures are usually due to any one or a combination of the following causes: defective design, poor material, and faulty execution.

The defects in a design may be many and various. The computations and assumptions on which they are based may be faulty and contrary to the established principles of statics and mechanics; the unit stresses used may be excessive, or the details of the design defective.

The design of reinforced concrete structures should receive at least the same careful consideration as those of steel, and only engineers with sufficient experience and good judgment should be intrusted with such work.

The computations should include all minor details, which are sometimes of the utmost importance. The design should show clearly the size and position of the reinforcement, and should provide for proper connections between the component parts, so that they cannot be displaced. As the connections between reinforced concrete members are frequently a source of weakness, the design should include a detailed study of such connections, accompanied by computations to prove their strength.

The use of high unit stresses, approaching the danger line, is a defect in the design of reinforced concrete structures.

Articulated concrete structures, designed in imitation of steel trusses, may be mentioned as illustrating a questionable use of reinforced concrete.

Poor material is sometimes used for the concrete, as well as for the reinforcement. The use of inferior concrete is generally due to a lack of experience of the contractor and his superintendents, or to the absence of proper supervision.

An unsuitable quality of steel for reinforcement is sometimes prescribed in specifications, for the purpose of reducing the cost. For steel structures, a high grade of material is specified, while the steel used for reinforcing concrete is sometimes made of unsuitable, brittle material.

Faulty execution and careless workmanship may generally be attributed to unintelligent or insufficient supervision.

While other engineering structures, upon the safety of which human lives depend, are generally designed by engineers employed by the

owner, and the contracts let on the engineer's design and specifications, in accordance with legitimate practice, reinforced concrete structures frequently are designed by contractors or by engineers commercially interested, and the contract let for a lump sum, regardless of the merits of the design.

The construction of buildings in large cities is regulated by ordinances or building laws, and the work is inspected by municipal authorities. For reinforced concrete work, however, the limited supervision which municipal inspectors are able to give is not sufficient. Means for more adequate supervision and inspection should, therefore, be provided.

3. Responsibility and Supervision.

The execution of the work should not be separated from the design, since intelligent supervision and successful execution can be expected only when both functions are combined. The engineer who prepares the design and specifications should therefore have the supervision of the execution of the work.

The Committee recommends the following rules for structures of reinforced concrete, for the purpose of fixing the responsibility and providing for adequate supervision during construction:

a. Before work is commenced, complete plans shall be prepared, accompanied by specifications, static computations and descriptions showing the general arrangement and all details. The static computations shall give the loads assumed separately, such as dead and live loads, wind and impact, if any, and the resulting stresses.

b. The specifications shall state the qualities of the materials to be used for making the concrete, and the manner in which they are to be proportioned.

c. The strength which the concrete is expected to attain after a definite period shall be stated in the specifications.

d. The drawings and specifications shall be signed by the engineer and the contractor.

e. The approval of plans and specifications by other authorities shall not relieve the engineer nor the contractor of responsibility.

f. Inspection during construction shall be made by competent inspectors employed by, and under the supervision of, the engineer, and shall cover the following:

1. The materials.
2. The correct construction and erection of the forms and the supports.
3. The sizes, shapes and arrangement of the reinforcement.
4. The proportioning, mixing and placing of the concrete.
5. The strength of the concrete by tests of standard test pieces made on the work.

6. Whether the concrete is sufficiently hardened before the forms and supports are removed.
7. Prevention of injury to any part of the structure by and after the removal of the forms.
8. Comparison of dimensions of all parts of the finished structure with the plans.

g. Load tests on portions of the finished structure shall be made where there is reasonable suspicion that the work has not been properly performed, or that, through influences of some kind, the strength has been impaired. Loading shall be carried to such a point that twice the calculated working stresses in critical parts are reached, and such loads shall cause no permanent deformations. Load tests shall not be made until after 60 days of hardening.

4. Destructive Agencies.

a. Corrosion of Metal Reinforcement.—Tests and experience have proved that steel embedded in good concrete will not corrode, no matter whether located above or below fresh or sea water level. If the concrete is porous, so as to be readily permeable to water, as, where the concrete is laid with a very dry consistency, the metal may be corroded in the presence of moisture.

b. Electrolysis.—There is little accurate information available as to the effect of electrolysis on concrete. The few experiments that are available seem to indicate that concrete may be damaged through the leakage of small electrical currents through the mass, particularly where steel is embedded in the concrete. These experiments are not conclusive, however, and the large numbers of reinforced concrete structures subject to the action of electrolysis, in which the metal and concrete are in perfect condition, would seem to indicate that the destructive action reported was due to abnormal conditions which do not often occur in practice.

c. Salt Water.—The data available concerning the effect of sea water on concrete or reinforced concrete are inconclusive and limited in amount. There have been no authentic cases reported where the disintegration has proved to be due entirely to sea water. The decomposition that has been reported manifests itself in a number of ways; in some cases the mortar softens and crumbles; in others, a crust forms which in time comes off. It has been found, however, that where concrete is proportioned in such a way as to secure a maximum density and is mixed thoroughly it makes an impervious concrete, upon which sea water has apparently little effect. Sea-walls have been standing for considerable lengths of time without apparent injury. In many of our harbors where the water has been rendered brackish through the rivers discharging into them, the action that has been reported has been at the water line and was probably due in part to freezing.

d. Acids.—Concrete of first-class quality, thoroughly hardened, is affected appreciably only by strong acids which seriously injure other materials. A substance like manure, because of the acid in its composition, is injurious to green concrete, but after the concrete has thoroughly hardened it satisfactorily resists such action.

e. Oils.—When concrete is properly made and the surface carefully finished and hardened it resists the action of such oils as petroleum and ordinary engine oils. Certain oils which contain fatty acids appear to produce injurious effects.

f. Alkalies.—The action of alkalies on concrete is problematical. In the reclamation of arid land, where the soil is heavily charged with alkaline salts, it has been found that concrete, stone, brick, iron and other materials are injured under certain conditions. It would seem that at the level of the ground-water such structures are disintegrated, possibly due in part to the effect of formation of crystals resulting from the alternate wetting and drying of the surface of the concrete at this ground-water line. Such destructive action can be prevented by the use of an insulating coating.

III. Materials.

A knowledge of the properties of the materials entering into concrete and reinforced concrete is the first essential. The importance of the quality of the materials used cannot be over-estimated, and, not only the cement, but also the aggregates, should be subject to such definite requirements and tests as will insure a concrete of the required quality.

1. Cement.

There are available, for construction purposes, Portland, Natural, and Puzzolan or Slag cements. Only Portland cement is suitable for reinforced concrete.

a. Portland Cement is the finely pulverized product resulting from the calcination to incipient fusion of an intimate mixture of properly proportioned argillaceous and calcareous materials. It has a definite chemical composition varying within comparatively narrow limits.

Portland cement should be used in reinforced concrete construction and any construction that will be subject to shocks or vibrations or stresses other than direct compression.

b. Natural Cement is the finely pulverized product resulting from the calcination of an argillaceous limestone at a temperature only sufficient to drive off the carbonic acid gas. While the limestone must have a certain composition, this composition may vary in much wider limits than in the case of Portland cement. Natural cement does not develop its strength as quickly nor is it as uniform in composition as Portland cement.

Natural cement may be used in massive masonry where weight rather than strength is the essential feature.

Where economy is the governing factor, a comparison may be made between the use of Natural cement and a leaner mixture of Portland cement that will develop the same strength.

c. Puzzolan or Slag Cement is the finely pulverized product resulting from grinding a mechanical mixture of granulated basic blast-furnace slag and hydrated lime.

Puzzolan cement is not nearly as strong, uniform, or reliable as Portland or Natural cement, is not extensively used, and never in important work; it should be used only for foundation work underground where it is not exposed to air or running water.

d. Specifications. The cement should meet the requirements of the Standard Specifications for Cement (see Appendix, p. 454). A number of societies have been working on methods for testing and specifications for cement. The best practice seems to be represented in the standard methods of testing and specifications for cement which are the result of the joint labors of Special Committees of the American Society of Civil Engineers, American Society for Testing Materials, American Railway Engineering and Maintenance of Way Association, American Institute of Architects, and others.

2. Aggregates.

Extreme care should be exercised in selecting the aggregates for mortar and concrete, and careful tests made of the materials for the purpose of determining their qualities and the grading necessary to secure maximum density* or a minimum percentage of voids.

a. Fine Aggregate consists of sand, crushed stone, or gravel screenings, passing when dry a screen having ¼-in. diameter holes. It should be preferably of silicious material, clean, coarse, free from vegetable loam or other deleterious matter.

A gradation of the grain from fine to coarse is generally advantageous.

Mortars composed of one part Portland cement and three parts fine aggregate, by weight, when made into briquettes should show a tensile strength of at least 70% of the strength of 1:3 mortar of the same consistency made with the same cement and standard Ottawa sand. To avoid the removal of any coating on the grains, which may affect the strength, bank sands should not be dried before being made into mortar, but should contain natural moisture. The percentage of moisture may be determined upon a separate sample for correcting weight. From 10 to 40% more water may be required in mixing bank or artificial sands than for standard Ottawa sand to produce the same consistency.

b. Coarse Aggregate consists of inert material, such as crushed stone, or gravel, which is retained on a screen having ¼-in. diameter

* A convenient coefficient of density is the ratio of the sum of the volumes of materials contained in a unit volume to the total unit volume.

holes. The particles should be clean, hard, durable, and free from all deleterious material. Aggregates containing soft, flat or elongated particles should be excluded from important structures. A gradation of sizes of the particles is generally advantageous.

The maximum size of the coarse aggregate shall be such that it will not separate from the mortar in laying and will not prevent the concrete from fully surrounding the reinforcement and filling all parts of the forms. Where concrete is used in mass, the size of the coarse aggregate may be such as to pass a 3-in. ring. For reinforced members a size to pass a 1-in. ring, or a smaller size, may be used.

Cinder concrete is not suitable for reinforced concrete structures, and may be safely used only in mass for very light loads or for fireproofing.

Where cinder concrete is permissible, the cinders used as the coarse aggregate should be composed of hard, clean, vitreous clinker, free from sulphides, unburned coal, or ashes.

3. Water.

The water used in mixing concrete should be free from oil, acid, strong alkalies, or vegetable matter.

4. Metal Reinforcement.

The Committee recommends, as a suitable material for reinforcement, steel filling the requirements of the specifications adopted by the American Railway Engineering and Maintenance of Way Association (Appendix, p. 458).

For the reinforcement of slabs, small beams, or minor details, or for the prevention of shrinkage cracks where wire or small rods are suitable, material conforming to the requirements of either Specification A or B given in the Appendix (pages 459 and 460) may be used.

The reinforcement should be free from rust, scale, or coatings of any character which would tend to reduce or destroy the bond.

IV. Preparation and Placing of Mortar and Concrete.

1. Proportions.

The materials to be used in concrete should be carefully selected, of uniform quality, and proportioned with a view to securing as nearly as possible a maximum density.

a. Unit of Measure.—The unit of measure should be the barrel, which should be taken as containing 3.8 cu. ft. Four bags containing 94 lb. of cement each should be considered the equivalent of one barrel. Fine and coarse aggregate should be measured separately as loosely thrown into the measuring receptacle.

b. Relation of Fine and Coarse Aggregates.—The fine and coarse aggregates should be used in such relative proportions as will insure maximum density. In unimportant work it is sufficient to do this by

individual judgment, using correspondingly higher proportions of cement; for important work these proportions should be carefully determined by density experiments, and the sizing of the fine and coarse aggregates should be uniformly maintained, or the proportions changed to meet the varying sizes.

c. Relation of Cement and Aggregates.—For reinforced concrete construction, a density proportion based on 1:6 should generally be used, *i. e.*, one part of cement to a total of six parts of fine and coarse aggregates measured separately.

In columns, richer mixtures are often required, while for massive masonry or rubble concrete a leaner mixture, of 1:9 or even 1:12, may be used. These proportions should be determined by the strength or wearing qualities required in the construction at the critical period of its use. Experienced judgment based on individual observation and tests of similar conditions in similar localities is the best guide as to the proper proportions for any particular case.

2. Mixing.

The ingredients of concrete should be thoroughly mixed to the desired consistency, and the mixing should continue until the cement is uniformly distributed and the mass is uniform in color and homogeneous, since the maximum density and therefore the greatest strength of a given mixture depends largely on thorough and complete mixing.

a. Measuring Ingredients.—Methods of measurement of the proportions of the various ingredients, including the water, should be used, which will secure separate uniform measurements at all times.

b. Machine Mixing.—When the conditions will permit, a machine mixer of a type which insures the uniform proportioning of the materials throughout the mass should be used, since a more thorough and uniform consistency can be thus obtained.

c. Hand Mixing.—When it is necessary to mix by hand, the mixing should be on a water-tight platform, and especial precautions should be taken to turn the materials until they are homogeneous in appearance and color.

d. Consistency.—The materials should be mixed wet enough to produce a concrete of such a consistency as will flow into the forms and about the metal reinforcement, and, at the same time, can be conveyed from the mixer to the forms without separation of the coarse aggregate from the mortar.

e. Retempering.—Retempering mortar or concrete, *i. e.*, remixing with water after it has partially set, should not be permitted.

3. Placing of Concrete.

a. Methods.—Concrete, after the addition of water to the mix, should be handled rapidly, and in as small masses as is practicable, from the place of mixing to the place of final deposit, and under no

circumstances should concrete be used that has partially set before final placing. A slow-setting cement should be used when a long time is likely to occur between mixing and final placing.

The concrete should be deposited in such a manner as will permit the most thorough compacting, such as can be obtained by working with a straight shovel or slicing tool kept moving up and down until all the ingredients have settled in their proper place by gravity and the surplus water has been forced to the surface.

In depositing the concrete under water, special care should be exercised to prevent the cement from being floated away, and to prevent the formation of laitance which hardens very slowly and forms a poor surface on which to deposit fresh concrete. Laitance is formed in both still and running water, and should be removed before placing fresh concrete.

Before placing the concrete, care should be taken to see that the forms are substantial and thoroughly wetted and the space to be occupied by the concrete is free from débris. When the placing of the concrete is suspended, all necessary grooves for joining future work should be made before the concrete has had time to set.

When work is resumed, concrete previously placed should be roughened, thoroughly cleansed of foreign material and laitance, drenched and slushed with a mortar consisting of one part Portland cement and not more than two parts fine aggregate.

The faces of concrete exposed to premature drying should be kept wet for a period of at least seven days.

b. Freezing Weather.—Concrete for reinforced structures should not be mixed or deposited at a freezing temperature, unless special precautions are taken to avoid the use of materials containing frost or covered with ice crystals, and to provide means to prevent the concrete from freezing after being placed in position and until it has thoroughly hardened.

c. Rubble Concrete.—Where the concrete is to be deposited in massive work, its value may be improved and its cost materially reduced through the use of clean stones thoroughly embedded in the concrete as near together as is possible and still entirely surrounded by concrete.

V. Forms.

Forms should be substantial and unyielding, so that the concrete shall conform to the designed dimensions and contours, and should be tight to prevent the leakage of mortar.

The time for removal of forms is one of the most important steps in the erection of a structure of concrete or reinforced concrete. Care should be taken to inspect the concrete and ascertain its hardness before removing the forms.

So many conditions affect the hardening of concrete that the proper time for the removal of the forms should be decided by some com-

petent and responsible person, especially where the atmospheric conditions are unfavorable.

VI. Details of Construction.

1. Joints.

a. Reinforcement.—Wherever in tension reinforcement it is necessary to splice the reinforcing bars, the length of lap shall be determined on the basis of the safe bond stress and the stress in the bar at the point of splice; or a connection shall be made between the bars of sufficient strength to carry the stress. Splices at points of maximum stress should be avoided. In columns, large bars should be properly butted and spliced; small bars may be treated as indicated for tension reinforcement, or their stress may be taken off by being embedded in large masses of concrete. At foundations, bearing plates should be provided for large bars or structural forms.

b. Concrete.—For concrete construction it is desirable to cast the entire structure at one operation, but as this is not always possible, especially in large structures, it is necessary to stop the work at some convenient point. This point should be selected so that the resulting joint may have the least possible effect on the strength of the structure. It is therefore recommended that the joint in columns be made flush with the lower side of the girders; that the joints in girders be at a point midway between supports, but should a beam intersect a girder at this point, the joint should be offset a distance equal to twice the width of the beam; that the joints in the members of a floor system should in general be made at or near the center of the span.

Joints in columns should be perpendicular to the axis of the column, and in girders, beams and floor slabs perpendicular to the plane of their surfaces.

2. Shrinkage.

Girders should never be constructed over freshly formed columns without permitting a period of at least two hours to elapse, thus providing for settlement or shrinkage in the columns. Before resuming work, the top of the column should be thoroughly cleansed of foreign matter and laitance. If the concrete in the column has become hard, the top should also be drenched and slushed with a mortar consisting of one part Portland cement and not more than two parts fine aggregate before placing additional concrete.

3. Temperature Changes.

Concrete is sensitive to temperature changes, and it is necessary to take this fact into account in designing and erecting concrete structures. In some positions the concrete is subjected to a much greater fluctuation in temperature than in others, and in such cases joints are necessary. The frequency of these joints will depend, first, upon the range of temperature to which concrete will be subjected; second, upon

the quantity and position of the reinforcement. These points should be determined and provided for in the design. In massive work, such as retaining walls, abutments, etc., built without reinforcement, joints should be provided, approximately, every 50 ft. throughout the length of the structure. To provide against the structures being thrown out of line by unequal settlement, each section of the wall may be tongued and grooved into the adjoining section. To provide against unsightly cracks, due to unequal settlement, a joint should be made at all sharp angles.

4. Fire-Proofing.

The actual fire tests of concrete and reinforced concrete have been limited, but experience, together with the results of tests so far made, indicate that concrete may be safely used for fire-proofing purposes. Concrete itself is incombustible and reasonably proof against fire when composed of a silicious sand and a hard coarse aggregate such as igneous rock.

For a fire-proof covering, these same materials may be used, or clean hard-burned cinders may be substituted for the coarse aggregate.

The low rate of heat conductivity of concrete is one reason of its value for fire-proofing. The dehydration of the water of crystallization of concrete probably begins at about 500° Fahr., and is completed at about 900° Fahr., but experience indicates that the volatilization of the water absorbs heat from the surrounding mass, which, together with the resistance of the air cells, tends to increase the heat resistance of the concrete, so that the process of dehydration is very much retarded. The concrete that is actually affected by fire remains in position and affords protection to the concrete beneath it.

It is recommended that in monolithic concrete columns, the concrete to a depth of 1½ in. be considered as protective covering and not included in the effective section.

The thickness of the protective coating required depends upon the probable duration of a fire which is likely to occur in the structure, and should be based on the rate of heat conductivity. The question of the conductivity of concrete is one which requires further study and investigation before a definite rate for different classes of concrete can be fully established. However, for ordinary conditions it is recommended that the metal in girders and columns be protected by a minimum of 2 in. of concrete; that the metal in beams be protected by a minimum of 1½ in. of concrete, and that the metal in floor slabs be protected by a minimum of 1 in. of concrete.

It is recommended that the corners of columns, girders and beams be beveled or rounded, as a sharp corner is more seriously affected by fire than a round one.

5. Water-Proofing.

Many expedients have been used to render concrete impervious to water under normal conditions, and also under pressure conditions that

exist in reservoirs, dams, and conduits of various kinds. Experience shows, however, that where mortar or concrete is proportioned to obtain the greatest practicable density and is mixed to a rather wet consistency, the resulting mortar or concrete is impervious under ordinary conditions. A concrete of dry consistency is more or less pervious to water, and compounds of various kinds have been mixed with the concrete, or applied as a wash to the surface for the purpose of making it water-tight. Many of these compounds are of but temporary value, and in time lose their power of imparting impermeability to the concrete.

In the case of subways, long retaining walls, and reservoirs, leakage cracks may be prevented by horizontal and vertical reinforcement, properly proportioned and located, provided the concrete itself is impervious.

Such reinforcement distributes the stretch due to contraction or settlement so that the cracks are too minute to permit leakage, or are soon closed by infiltration of silt.

Asphaltic or coal-tar preparations, applied either as a mastic or as a coating on felt or cloth fabric, are used for water-proofing, and should be proof against injury by liquids or gases.

6. Surface Finish.

Concrete is a material of an individual type, and should not be used in imitation of other structural materials. One of the important problems connected with the use of concrete is the character of the finish of exposed surfaces. The finish of the surface should be determined before the concrete is placed, and the work conducted so as to make possible the finish desired. For many forms of construction the natural surface of the concrete is unobjectionable, but frequently the marks of the boards and the flat dead surface are displeasing, and make some special treatment desirable. A treatment of the surface which removes the film of mortar and brings the coarser particles of the concrete into relief is frequently used to remove the form markings, break the monotonous appearance of the surface, and make it more pleasing. Plastering of surfaces should be avoided, for the other methods of treatment are more reliable and usually much more satisfactory. Plastering, even if carefully applied, is likely to peel off under the action of frost or temperature changes.

VII. Design.

1. Massive Concrete.

In the design of massive concrete or plain concrete, no account should be taken of the tensile strength of the material, and sections should usually be so proportioned as to avoid tensile stresses. This will generally be accomplished, in the case of rectangular shapes, if the line of pressure is kept within the middle third of the section, but in very

large structures, such as high masonry dams, a more exact analysis may be required. Structures of massive concrete are able to resist unbalanced lateral forces by reason of their weight, hence the element of weight rather than strength often determines the design. A relatively cheap and weak concrete will therefore often be suitable for massive concrete structures. Owing to its low extensibility, the contraction due to hardening and to temperature changes requires special consideration, and, except in the case of very massive walls such as dams, it is desirable to provide joints at intervals to localize the effect of such contraction. The spacing of such joints will depend upon the form and dimensions of the structure and its degree of exposure.

Massive concrete may well be used for piers and short columns, in which the ratio of length to least width is relatively small. Under ordinary conditions this ratio should not exceed six, but, where the central application of the load is assured, a somewhat higher value may safely be used.

Massive concrete is also a suitable material for arches of moderate span where the conditions as to foundations are favorable.

2. Reinforced Concrete.

By the use of metal reinforcement to resist the principal tensile stresses, concrete becomes available for general use in a great variety of structures and structural forms. This combination of concrete and steel is particularly advantageous in the beam, where both compression and tension exist; it is also advantageous in the column, where the main stresses are compressive, but where cross-bending may exist. The theory of design will therefore relate mainly to the analysis of beams and columns.

3. General Assumptions.

a. Loads.—The loads or forces to be resisted consist of:

1. The dead load, which includes the weight of the structure and fixed loads and forces.
2. The live load, or the loads and forces which are variable. The dynamic effect of the live load will often require consideration. Any allowance for the dynamic effect is preferably taken into account by adding the desired amount to the live load or to the live-load stresses. The working stresses hereinafter recommended are intended to apply to the equivalent static stresses so determined.

   In the case of high buildings the live load on columns may be reduced in accordance with the usual practice.

b. Lengths of Beams and Columns.—The span length for beams and slabs shall be taken as the distance from center to center of supports, but shall not be taken to exceed the clear span plus the depth of beam or slab. Brackets shall not be considered as reducing the clear span in the sense here intended.

The length of columns shall be taken as the maximum unsupported length.

c. Internal Stresses.—As a basis for calculations relating to the strength of structures, the following assumptions are recommended:

1. Calculations should be made with reference to working stresses and safe loads rather than with reference to ultimate strength and ultimate loads.
2. A plane section before bending remains plane after bending.
3. The modulus of elasticity of concrete in compression, within the usual limits of working stresses, is constant. The distribution of compressive stresses in beams is therefore rectilinear.
4. In calculating the moment of resistance of beams the tensile stresses in the concrete shall be neglected.
5. Perfect adhesion is assumed between concrete and reinforcement. Under compressive stresses the two materials are therefore stressed in proportion to their moduli of elasticity.
6. The ratio of the modulus of elasticity of steel to the modulus of elasticity of concrete may be taken at 15.
7. Initial stress in the reinforcement due to contraction or expansion in the concrete may be neglected.

It is appreciated that the assumptions herein given are not entirely borne out by experimental data. They are given in the interest of simplicity and uniformity, and variations from exact conditions are taken into account in the selection of formulas and working stresses.

For calculations relative to deflections, the tensile strength of the concrete should be taken into account. For such calculations, also, a value of 8 to 12 for the ratio of the moduli corresponds more nearly to the actual conditions and may well be used.

4. Tee-Beams.

In beam and slab construction, an effective bond should be provided at the junction of the beam and slab. When the principal slab reinforcement is parallel to the beam, transverse reinforcement should be used extending over the beam and well into the slab.

Where adequate bond between slab and web of beam is provided, the slab may be considered as an integral part of the beam, but its effective width shall be determined by the following rules:

a. It shall not exceed one-fourth of the span length of the beam;

b. Its overhanging width on either side of the web shall not exceed 4 times the thickness of the slab.

In the design of T-beams acting as continuous beams, due consideration should be given to the compressive stresses at the support.

5. Floor Slabs.

Floor slabs should be designed and reinforced as continuous over the supports. If the length of the slab exceeds 1.5 times its width the entire load should be carried by transverse reinforcement. Square slabs may well be reinforced in both directions.*

The loads carried to beams by slabs which are reinforced in two directions will not be uniformly distributed to the supporting beam, and may be assumed to vary in accordance with the ordinates of a triangle. The moments in the beams should be calculated accordingly.

6. Continuous Beams and Slabs.

When the beam or slab is continuous over its supports, reinforcement should be fully provided at points of negative moment. In computing the positive and negative moments in beams and slabs continuous over several supports, due to uniformly distributed loads, the following rules are recommended:

a. That for floor slabs the bending moments at center and at support be taken at $\frac{w\,l^2}{12}$ for both dead and live loads, where w represents the load per linear foot and l the span length.

b. That for beams the bending moment at center and at support for interior spans be taken at $\frac{w\,l^2}{12}$, and for end spans it be taken at $\frac{w\,l^2}{10}$ for center and adjoining support for both dead and live loads.

---

* The exact distribution of load on square and rectangular slabs, supported on four sides and reinforced in both directions cannot readily be determined. The following method of calculation is recognized to be faulty, but it is offered as a tentative method which will give results on the safe side. The distribution of load is first to be determined by the formula:

$$r = \frac{l^4}{l^4 + b^4}$$

in which r = proportion of load carried by the transverse reinforcement, l = length and b = breadth of slab. For various ratios of l/b the values of r are as follows:

l/b	r
1	0.50
1.1	0.59
1.2	0.67
1.3	0.75
1.4	0.80
1.5	0.83

Using the values above specified, each set of reinforcement is to be calculated in the same manner as slabs having supports on two sides only, but the total amount of reinforcement thus determined may be reduced 25% by gradually increasing the rod spacing from the third point to the edge of the slab.

In the case of beams and slabs continuous for two spans only, or of spans of unusual length, more exact calculations should be made. Special consideration is also required in the case of concentrated loads.

Where beams are reinforced on the compression side, the steel may be assumed to carry its proportion of stress in accordance with the provisions of Chapter VII, Section 3, *c*, Paragraph 6. In the case of continuous beams, tensile and compressive reinforcement over supports must extend sufficiently beyond the support to develop the requisite bond strength.

7. Bond Strength and Spacing of Bars.

Adequate bond strength should be provided in accordance with the formula hereinafter given. Where a portion of the bars is bent up near the end of a beam, the bond stress in the remaining straight bars will be less than is represented by the theoretical formula.

Where high bond resistance is required, the deformed bar is a suitable means of supplying the necessary strength. Adequate bond strength throughout the length of a bar is preferable to end anchorage, but such anchorage may properly be used in special cases. Anchorage furnished by short bends at a right angle is less effective than hooks consisting of turns through 180 degrees.

The lateral spacing of parallel bars should not be less than two and one-half diameters, center to center, nor should the distance from the side of the beam to the center of the nearest bar be less than two diameters. The clear spacing between two layers of bars should not be less than $\frac{1}{2}$ in.

8. Shear and Diagonal Tension.

Calculations for web resistance shall be made on the basis of maximum shearing stress as determined by the formulas hereinafter given. When the maximum shearing stresses exceed the value allowed for the concrete alone, web reinforcement must be provided to aid in carrying the diagonal tension stresses. This web reinforcement may consist of bent bars, or inclined or vertical members attached to or looped about the horizontal reinforcement. Where inclined members are used, the connection to the horizontal reinforcement shall be such as to insure against slip.

Experiments bearing on the design of details of web reinforcement are not yet complete enough to allow more than general and tentative recommendations to be made. It is well established, however, that a very moderate amount of reinforcement, such as is furnished by a few bars bent up at small inclination, increases the strength of a beam against failure by diagonal tension to a considerable degree; and that a sufficient amount of web reinforcement can readily be provided to increase the shearing resistance to a value from three or more

times that found when the bars are all horizontal and no web reinforcement is used. The following allowable values for the maximum shearing stress are therefore recommended, based on the working stresses of Section VIII, page 452.

a. For beams with horizontal bars, only 40 lb. per sq. in.

b. For beams in which a part of the horizontal reinforcement is used in the form of bent-up bars, arranged with due respect to the shearing stresses, a higher value may be allowed, but not to exceed 60 lb. per sq. in.

c. For beams thoroughly reinforced for shear, a value not exceeding 120 lb. per sq. in.

In the calculation of web reinforcement to provide the strength required under *c* above, the concrete may be counted upon as carrying one-third of the shear. The remainder is to be provided for by means of metal reinforcement consisting of bent rods or stirrups, but preferably both. The requisite amount of such reinforcement may be estimated on the assumption that the entire shear on a section, less the amount assumed to be carried by the concrete, is carried by the reinforcement in a length of beam equal to its depth.

The longitudinal spacing of stirrups or bent rods shall not exceed three-fourths the depth of the beam.

It is important that adequate bond strength be provided to develop fully the assumed strength of all shear reinforcement.

Inasmuch as small deformations in the horizontal steel tends to prevent the formation of diagonal cracks, a beam will be strengthened against diagonal tension failure by so arranging the horizontal reinforcement that the unit stresses at points of large shear shall be relatively low.

9. Columns.

It is recommended that the ratio of the unsupported length of a column to its least width be limited to 15.

The effective area of the column shall be taken as the area within the protective covering, as defined in Chapter VI, Section 4; or, in the case of hooped columns or columns reinforced with structural shapes, it shall be taken as the area within the hooping or structural shapes.

Columns may be reinforced by means of longitudinal rods, by bands or hoops, by bands or hoops together with longitudinal bars, or by structural forms which in themselves are sufficiently rigid to act as columns. The general effect of bands or hoops is to increase greatly the "toughness" of the column and its ultimate strength, but hooping has little effect upon its behavior within the limit of elasticity. It thus renders the concrete a safer and more reliable material, and should permit the use of a somewhat higher working stress. The beneficial

effects of "toughening" are adequately provided by a moderate amount of hooping, a larger amount serving mainly to increase the ultimate strength and the possible deformation before ultimate failure.

The following recommendations are made for the relative working stresses in the concrete for the several types of columns:

a. Columns with longitudinal reinforcement only, the unit stress recommended for axial compression in Chapter VIII, Section 3.

b. Columns with reinforcement of bands or hoops, as hereinafter specified, stresses 20% higher than given for *a*.

c. Columns reinforced with not less than 1% and not more than 4% of longitudinal bars and with bands or hoops, stresses 45% higher than given for *a*.

d. Columns reinforced with structural steel column units which thoroughly encase the concrete core, stresses 45% higher than given for *a*.

In all cases, longitudinal steel is assumed to carry its proportion of stress in accordance with Section 3. The hoops or bands are not to be counted upon directly as adding to the strength of the column.

Bars composing longitudinal reinforcement shall be straight, and shall have sufficient lateral support to be securely held in place until the concrete has set.

Where bands or hoops are used, the total amount of such reinforcement shall not be less than 1% of the volume of the column enclosed. The clear spacing of such bands or hoops shall not be greater than one-fourth the diameter of the enclosed column. Adequate means must be provided to hold bands or hoops in place so as to form a column, the core of which shall be straight and well centered.

Bending stresses due to eccentric loads must be provided for by increasing the section until the maximum stress does not exceed the values above specified.

10. Reinforcing for Shrinkage and Temperature Stresses.

Where large areas of concrete are exposed to atmospheric conditions, the changes of form due to shrinkage (resulting from hardening) and to action of temperature are such that large cracks will occur in the mass, unless precautions are taken to so distribute the stresses as either to prevent the cracks altogether or to render them very small. The size of the cracks will be directly proportional to the diameter of the reinforcing bars and inversely proportional to the percentage of reinforcement and also to its bond resistance per unit of surface area. To be most effective, therefore, reinforcement should be placed near the surface and well distributed, and a form of reinforcement used which will develop a high bond resistance.

VIII. Working Stresses.

1. General Assumptions.

The following working stresses are recommended for static loads. Proper allowances for vibration and impact are to be added to live loads where necessary to produce an equivalent static load before applying the unit stresses in proportioning parts.

In selecting the permissible working stress to be allowed on concrete, we should be guided by the working stresses usually allowed for other materials of construction, so that all structures of the same class but composed of different materials may have approximately the same degree of safety.

The stresses for concrete are proposed for concrete composed of one part Portland cement and six parts of aggregates, capable of developing an average compressive strength of 2 000 lb. per sq. in. at 28 days, when tested in cylinders 8 in. in diameter and 16 in. long, under laboratory conditions of manufacture and storage, using the same consistency as is used in the field. In considering the factors recommended with relation to this strength, it is to be borne in mind that the strength at 28 days is by no means the ultimate which will be developed at a longer period, and therefore they do not correspond with the real factor of safety. On concretes, in which the material of the aggregate is inferior, all stresses should be proportionally reduced, and similar reduction should be made when leaner mixes are to be used. On the other hand, if, with the best quality of aggregates, the richness is increased, an increase may be made in all working stresses proportional to the increase in compressive strength at 28 days, but this increase shall not exceed 25 per cent.

2. Bearing.

When compression is applied to a surface of concrete larger than the loaded area, a stress of 32.5% of the compressive strength at 28 days, or 650 lb. per sq. in. on the above-described concrete, may be allowed. This pressure is probably unnecessarily low when the ratio of the stressed area to the whole area of the concrete is much below unity, but is recommended for general use rather than a variable unit based upon this ratio.

3. Axial Compression.

For concentric compression on a plain concrete column or pier, the length of which does not exceed 12 diameters, 22.50% of the compressive strength at 28 days, or 450 lb. per sq. in. on 2 000-lb. concrete, may be allowed.

For other forms of columns, the stresses obtained from the ratios given in Chapter VII, Section 9, may govern.

4. Compression in Extreme Fiber.

The extreme fiber stress of a beam, calculated on the assumption of a constant modulus of elasticity for concrete under working stresses, may be allowed to reach 32.5% of the compressive strength at 28 days, or 650 lb. per sq. in. for 2 000-lb. concrete. Adjacent to the support of continuous beams, stresses 15% higher may be used.

5. Shear and Diagonal Tension.

Where pure shearing stress occurs, that is, uncombined with compression normal to the shearing surface, and with all tension normal to the shearing plane provided for reinforcement, a shearing stress of 6% of the compressive strength at 28 days, or 120 lb. per sq. in. on 2 000-lb. concrete, may be allowed. Where the shear is combined with an equal compression, as on a section of a column at 45° with the axis, the stress may equal one-half the compressive stress allowed. For ratios of compressive stress to shear intermediate between 0 and 1, proportionate shearing stresses shall be used.

In calculations on beams in which diagonal tension is considered to be taken by the concrete, the vertical shearing stresses should not exceed 2% of the compressive strength at 28 days, or 40 lb. per sq. in. for 2 000-lb. concrete.

6. Bond.

The bonding stress between concrete and plain reinforcing bars may be assumed at 4% of the compressive strength at 28 days, or 80 lb. per sq. in. for 2 000-lb. concrete; in the case of drawn wire, 2% or 40 lb. on 2 000-lb. concrete.

7. Reinforcement.

The tensile stress in steel should not exceed 16 000 lb. per sq. in. The compressive stress in reinforcing steel should not exceed 16 000 lb. per sq. in., or 15 times the working compressive stress in the concrete.

In structural steel members, the working stresses adopted by the American Railway Engineering and Maintenance of Way Association are recommended.

8. Modulus of Elasticity.

The value of the modulus of elasticity of concrete has a wide range, depending upon the materials used, the age, the range of stresses between which it is considered, as well as other conditions. It is recommended that in all computations it be assumed as one-fifteenth that of steel, as, while not rigorously accurate, this assumption will give safe results.

JANUARY, 1909.

RICHARD L. HUMPHREY,
Secretary,

W. K. HATT,
R. W. LESLEY,
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APPENDIX.

1.

STANDARD SPECIFICATIONS.

1.

Standard Specifications for Cement.

Adopted August 15th, 1908, by the
American Society for Testing Materials.

General Observations.

1. These remarks have been prepared with a view of pointing out the pertinent features of the various requirements and the precautions to be observed in the interpretation of the results of the tests.

2. The Committee would suggest that the acceptance or rejection under these specifications be based on tests made by an experienced person having the proper means for making the tests.

SPECIFIC GRAVITY.

3. Specific gravity is useful in detecting adulteration. The results of tests of specific gravity are not necessarily conclusive as an indication of the quality of a cement, but when in combination with the results of other tests may afford valuable indications.

FINENESS.

4. The sieves should be kept thoroughly dry.

TIME OF SETTING.

5. Great care should be exercised to maintain the test pieces under as uniform conditions as possible. A sudden change or wide range of temperature in the room in which the tests are made, a very dry or humid atmosphere, and other irregularities, vitally affect the rate of setting.

TENSILE STRENGTH.

6. Each consumer must fix the minimum requirements for tensile strength to suit his own conditions. They shall, however, be within the limits stated.

CONSTANCY OF VOLUME.

7. The tests for constancy of volume are divided into two classes, the first normal, the second accelerated. The latter should be regarded as a precautionary test only, and not infallible. So many conditions enter into the making and interpreting of it that it should be used with extreme care.

8. In making the pats the greatest care should be exercised to avoid

initial strains due to molding or to too rapid drying-out during the first twenty-four hours. The pats should be preserved under the most uniform conditions possible, and rapid changes of temperature should be avoided.

9. The failure to meet the requirements of the accelerated tests need not be sufficient cause for rejection. The cement may, however, be held for twenty-eight days, and a retest made at the end of that period using a new sample. Failure to meet the requirements at this time should be considered sufficient cause for rejection, although in the present state of our knowledge it cannot be said that such failure necessarily indicates unsoundness, nor can the cement be considered entirely satisfactory simply because it passes the tests.

SPECIFICATIONS.

General Conditions.

1. All cement shall be inspected.

2. Cement may be inspected either at the place of manufacture or on the work.

3. In order to allow ample time for inspecting and testing, the cement should be stored in a suitable weather-tight building having the floor properly blocked or raised from the ground.

4. The cement shall be stored in such a manner as to permit easy access for proper inspection and identification of each shipment.

5. Every facility shall be provided by the Contractor and a period of at least twelve days allowed for the inspection and necessary tests.

6. Cement shall be delivered in suitable packages with the brand and name of manufacturer plainly marked thereon.

7. A bag of cement shall contain 94 pounds of cement net. Each barrel of Portland cement shall contain 4 bags, and each barrel of natural cement shall contain 3 bags of the above net weight.

8. Cement failing to meet the seven-day requirements may be held awaiting the results of the twenty-eight day tests before rejection.

9. All tests shall be made in accordance with the methods proposed by the Committee on Uniform Tests of Cement of the American Society of Civil Engineers, presented to the Society January 21, 1903, and amended January 20, 1904, and January 15, 1908, with all subsequent amendments thereto. (See addendum to these specifications.)

10. The acceptance or rejection shall be based on the following requirements:

Natural Cement.

11. *Definition.* This term shall be applied to the finely pulverized product resulting from the calcination of an argillaceous limestone at a temperature only sufficient to drive off the carbonic acid gas.

FINENESS.

12. It shall leave by weight a residue of not more than 10 per cent. on the No. 100, and 30 per cent. on the No. 200 sieve.

TIME OF SETTING.

13. It shall not develop initial set in less than ten minutes, and shall not develop hard set in less than thirty minutes, or in more than three hours.

TENSILE STRENGTH.

14. The minimum requirements for tensile strength for briquettes one inch square in cross section shall be within the following limits, and shall show no retrogression in strength within the periods specified.*

Age.	*Neat Cement.*	*Strength.*
24 hours in moist air		50-100 lbs.
7 days (1 day in moist air, 6 days in water)		100-200 "
28 days (1 " " 27 " ")		200-300 "
	One Part Cement, Three Parts Standard Sand.	
7 days (1 day in moist air, 6 days in water)		25- 75 "
28 days (1 " " 27 " ")		75-150 "

If the minimum strength is not specified, the mean of the above values shall be taken as the minimum strength required.

CONSTANCY OF VOLUME.

15. Pats of neat cement about three inches in diameter, one-half inch thick at center, tapering to a thin edge, shall be kept in moist air for a period of twenty-four hours.

(*a*) A pat is then kept in air at normal temperature.

(*b*) Another is kept in water maintained as near 70° F. as practicable.

16. These pats are observed at intervals for at least 28 days, and, to satisfactorily pass the tests, should remain firm and hard and show no signs of distortion, checking, cracking or disintegrating.

PORTLAND CEMENT.

17. *Definition.* This term is applied to the finely pulverized product resulting from the calcination to incipient fusion of an intimate mixture of properly proportioned argillaceous and calcareous materials, and to which no addition greater than 3 per cent. has been made subsequent to calcination.

* For example, the minimum requirement for the twenty-four hour neat cement test should be some specified value within the limits of 50 and 100 pounds, and so on for each period stated.

SPECIFIC GRAVITY.

18. The specific gravity of the cement, ignited at a low red heat, shall be not less than 3.10, and the cement shall not show a loss on ignition of more than 4 per cent.

FINENESS.

19. It shall leave by weight a residue of not more than 8 per cent. on the No. 100, and not more than 25 per cent. on the No. 200 sieve.

TIME OF SETTING.

20. It shall not develop initial set in less than thirty minutes, and must develop hard set in not less than one hour, nor more than ten hours.

TENSILE STRENGTH.

21. The minimum requirements for tensile strength for briquettes one inch square in section shall be within the following limits, and shall show no retrogression in strength within the periods specified.*

Age.	*Neat Cement.*	*Strength.*
24 hours in moist air		150-200 lbs.
7 days (1 day in moist air, 6 days in water)		450-550 "
28 days (1 " " 27 " ")		550-650 "
	One Part Cement, Three Parts Standard Sand.	
7 days (1 day in moist air, 6 days in water)		150-200 lbs.
28 days (1 " " 27 " ")		200-300 "

If the minimum strength is not specified, the mean of the above values shall be taken as the minimum strength required.

CONSTANCY OF VOLUME.

22. Pats of neat cement about three inches in diameter, one-half inch thick at the center, and tapering to a thin edge, shall be kept in moist air for a period of twenty-four hours.

(*a*) A pat is then kept in air at normal temperature and observed at intervals for at least 28 days.

(*b*) Another pat is kept in water maintained as near 70° F. as practicable, and observed at intervals for at least 28 days.

(*c*) A third pat is exposed in any convenient way in an atmosphere of steam, above boiling water, in a loosely closed vessel for five hours.

23. These pats, to satisfactorily pass the requirements, shall remain firm and hard and show no signs of distortion, checking, cracking or disintegrating.

* For example, the minimum requirement for the twenty-four hour neat cement test should be some specified value within the limits of 150 and 200 pounds, and so on for each period stated.

SULPHURIC ACID AND MAGNESIA.

24. The cement shall not contain more than 1.75 per cent. of anhydrous sulphuric acid (SO_3), nor more than 4 per cent. of magnesia (MgO).

2.

1. Abstract Embodying Portions Applicable to Concrete Reinforcement from the Standard Specifications for Structural Steel.

Adopted March, 1906, by the American Railway Engineering and Maintenance of Way Association.

Process of Manufacture.

83. Steel shall be made by the open-hearth process.

Schedule of Requirements.

84. The chemical and physical properties shall conform to the following limits:

Elements Considered.	Structural Steel.	Rivet Steel.
Phosphorus, max.... Basic	0.04 per cent.	0.04 per cent.
Acid	0.08 "	0.04 "
Sulphur, maximum	0.05 "	0.04 "
Ultimate tensile strength.	Desired	Desired
Pounds per square inch	60 000	50 000
Elongation, minimum, percentage in 8 in.	1 500 000*	1 500 000
Fig. 1	Ult. tensile strength	Ult. tensile strength
Character of fracture	Silky	Silky
Cold bends without fracture	180° flat†	180° flat‡

* See paragraph 11. † See paragraphs 12, 13 and 14. ‡ See paragraph 15.

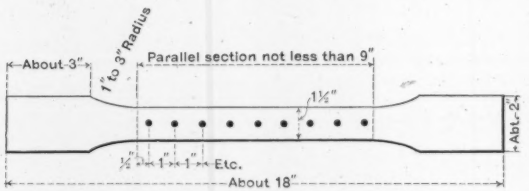


Fig. 1.

The yield point, as indicated by the drop of beam, shall be recorded in the test reports.

Allowable Variations.

85. If the ultimate strength varies more than 4 000 lb. from that desired, a retest shall be made on the same gauge, which, to be acceptable, shall be within 5 000 lb. of the desired ultimate.

Chemical Analyses.

86. Chemical determinations of the percentages of carbon, phosphorus, sulphur, and manganese shall be made by the manufacturer

from a test ingot taken at the time of the pouring of each melt of steel, and a correct copy of such analysis shall be furnished to the engineer or his inspector. Check analyses shall be made from finished material, if called for by the purchaser, in which case an excess of 25% above the required limits will be allowed.

92. At least one tensile and one bending test shall be made from each melt of steel as rolled. Number of Tests.

95. Full-sized material for steel 1 in. thick and over, tested as rolled, shall bend, cold, 180° around a pin, the diameter of which is equal to twice the thickness of the bar, without fracture on the outside of bend. Thick Material.

98. Finished material shall be free from injurious seams, flaws, cracks, defective edges or other defects, and have a smooth, uniform and workmanlike finish. Plates 36 in. in width and under shall have rolled edges. Finish.

99. Every finished piece of steel shall have the melt number and the name of the manufacturer stamped or rolled upon it. Stamping.

100. Material which, subsequent to the above tests at the mills, and its acceptance there, develops weak spots, brittleness, cracks, or other imperfections, or is found to have injurious defects, will be rejected at the shop and shall be replaced by the manufacturer at his own cost. Defective Material.

3.

ALTERNATIVE SPECIFICATIONS FOR METAL REINFORCEMENT.

A.

1. For wire and rod reinforcements, the steel may be manufactured from Bessemer billets (not re-rolled rails) and shall meet the following requirements:

2. *Tensile Tests.*

Tensile strength, in pounds per square inch, not less than 105 000.

Yield point, in pounds per square inch, not less than 52 500 nor more than 90% of the tensile strength.

Elongation in 8 in., not less than 10%, with the following modifications:

a. For each increase in diameter of $\frac{1}{8}$ in. above $\frac{3}{4}$ in. a deduction of 1% shall be made from the specified elongation.

b. For materials from $\frac{1}{4}$ in. to, but not including, $\frac{5}{16}$ in. diameter the elongation shall be 8 per cent.

For material $\frac{3}{16}$ in. to $\frac{1}{4}$ in., elongation 7 per cent.

For material $\frac{1}{8}$ in. to $\frac{3}{16}$ in., elongation 6 per cent.

For material less than $\frac{1}{8}$ in., elongation 5 per cent.

3. *Bending Tests.*

Test specimens for bending shall be bent, cold, under the following conditions, without fracture on the outside of the bent portion:

Around twice their own diameter:	Around their own diameter:
1 in. diameter, 80 degrees.	$\frac{1}{4}$ in. diameter, 130 degrees.
$\frac{3}{4}$ in. diameter, 90 degrees.	$\frac{3}{16}$ in. diameter, 140 degrees.
$\frac{1}{2}$ in. diameter, 110 degrees.	$\frac{1}{8}$ in. diameter or less, 180 degrees.

B.

Steel wire used for reinforcement should be drawn from bars of basic open-hearth steel of the same quality as that specified for rivet steel.

Test pieces of wire shall bend 180° around their own diameter without fracture.

SUGGESTED FORMULAS FOR REINFORCED CONCRETE CONSTRUCTION.

These formulas are based upon the assumptions and principles given in the chapter on Design.

a. Standard Notation.

1. *Rectangular Beams.*

The following notation is recommended:

f_s = tensile unit stress in steel.
f_c = compressive unit stress in concrete.
E_s = modulus of elasticity of steel.
E_c = modulus of elasticity of concrete.
$n = E_s \div E_c$.
M = moment of resistance, or bending moment in general.
A = steel area.
b = breadth of beam.
d = depth of beam to center of steel.
k = ratio of depth of neutral axis to effective depth, d.
z = depth of resultant compression below top.
j = ratio of lever arm of resisting couple to depth, d.
$jd = d - z$ = arm of resisting couple.
p = steel ratio (not percentage).

I-Beams.

b = width of flange.
b' = width of stem.
t = thickness of flange.

Beams Reinforced for Compression.

A' = area of compressive steel.
p' = steel ratio for compressive steel.

f_s' = unit compressive stress in steel.
C = total compressive stress in concrete.
C' = total compressive stress in steel.
d' = depth to center of compressive steel.
z = depth to resultant of C and C'.

Shear and Bond.

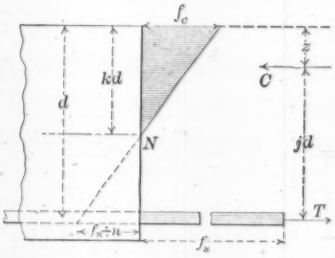
V = total shear.
v = shearing unit stress.
u = bond stress per unit area of bar.
o = circumference or perimeter of bar.
Σo = sum of the perimeters of all bars.

Columns.

A = total net area.
A_s = area of longitudinal steel.
A_c = area of concrete.
P = total safe load.

b. Formulas.

1. *Rectangular Beams.*



Position of neutral axis,

$$k = \sqrt{2pn + (pn)^2} - pn.$$

Arm of resisting couple,

$$j = 1 - \frac{1}{3}k.$$

[For f_s = 15 000 to 16 000 and f_c = 600 to 650, j may be taken at $\frac{7}{8}$.]

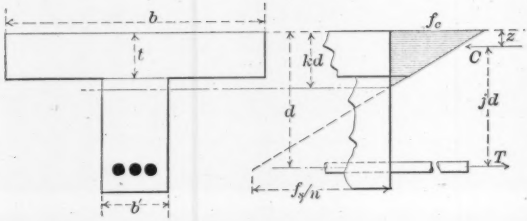
Fiber stresses,

$$f_s = \frac{M}{A\,j\,d} = \frac{M}{p\,j\,b\,d^2}. \quad f_c = \frac{2\,M}{j\,k\,b\,d^2} = \frac{2\,p\,f_s}{k}.$$

Steel ratio,

$$p = \frac{1}{2}\,\frac{1}{\frac{f_s}{f_c}\left(\frac{f_s}{n\,f_c} + 1\right)}.$$

2. T-Beams.



Case I. When the neutral axis lies in the flange, use the formulas for rectangular beams.

Case II. When the neutral axis lies in the stem.

The following formulas neglect the compression in the stem:

Position of neutral axis,

$$k\,d = \frac{2\,n\,d\,A + b\,t^2}{2\,n\,A + 2\,b\,t}.$$

Position of resultant compression,

$$z = \frac{3\,k\,d - 2\,t}{2\,k\,d - t} \cdot \frac{t}{3}.$$

Arm of resisting couple,

$$j\,d = d - z.$$

Fiber stresses,

$$f_s = \frac{M}{A\,j\,d}. \qquad f_c = \frac{M\,k\,d}{b\,t\,(k\,d - \frac{1}{2}\,t)\,j\,d} = \frac{f_s}{n} \cdot \frac{k}{1-k}.$$

(For approximate results, the formulas for rectangular beams may be used.)

The following formulas take into account the compression in the stem; they are recommended where the flange is small compared with the stem.

Position of neutral axis,

$$k\,d = \sqrt{\frac{2\,n\,d\,A + (b-b')\,t^2}{b'} + \left(\frac{n\,A + (b-b')\,t}{b'}\right)^2} - \frac{n\,A + (b-b')\,t}{b'}.$$

Position of resultant compression,

$$z = \frac{(k\,d\,t^2 - \frac{2}{3}\,t^3)\,b + (k\,d - t)^2\,[t + \frac{1}{3}\,(k\,d - t)]\,b'}{t\,(2\,k\,d - t)\,b + (k\,d - t)^2\,b'}.$$

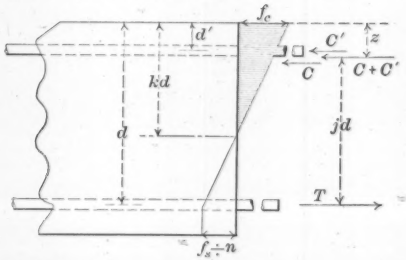
Arm of resisting couple,

$$j\,d = d - z.$$

Fiber stresses,

$$f_s = \frac{M}{A\,j\,d}. \qquad f_c = \frac{2\,M\,k\,d}{[(2\,k\,d - t)\,b\,t + (k\,d - t)^2\,b']\,j\,d}.$$

3. *Beams reinforced for compression,*



Position of neutral axis,

$$k = \sqrt{2\,n\left(p + p'\frac{d'}{d}\right) + n^2\,(p+p')^2} - n\,(p+p').$$

Position of resultant compression,

$$z = \frac{\frac{1}{3}\,k^3\,d + 2\,p'\,n\,d'\left(k - \frac{d'}{d}\right)}{k^2 + 2\,p'\,n\left(k - \frac{d'}{d}\right)}.$$

Arm of resisting couple,

$$j\,d = d - z.$$

Fiber stresses,

$$f_c = \frac{6\,M}{b\,d^2\left[3\,k - k^2 + \frac{6\,p'\,n}{k}\left(k - \frac{d'}{d}\right)\left(1 - \frac{d'}{d}\right)\right]}.$$

$$f_s = \frac{M}{p\,j\,b\,d^2} = n\,f\,\frac{1-k}{k}.$$

$$f_s' = n\,f_c\,\frac{k - \frac{d'}{d}}{k}.$$

4. *Shear, Bond, and Web Reinforcement.*

In the following formula, Σo, refers only to the bars constituting the tension reinforcement at the section in question and $j\,d$ is the lever arm of the resisting couple at the section.

For rectangular beams,

$$v = \frac{V}{b\,j\,d}.$$

$$u = \frac{V}{j\,d\,.\,\Sigma o}.$$

[For approximate results, j may be taken at $\frac{7}{8}$.]

The stresses in web reinforcement may be estimated by using the following formulas:

Vertical reinforcement,

$$P = \frac{V\,s}{j\,d}.$$

Reinforcement inclined at 45°,

$$P = 0.7\,\frac{V\,s}{j\,d},$$

in which P = stress in single reinforcing member, V = proportion of total shear assumed as carried by the reinforcement, and s = horizontal spacing of the reinforcing members.

The same formulas apply to beams reinforced for compression as regards shear and bond stress for tensile steel.

For T-beams,

$$v = \frac{V}{b'\,j\,d}, \quad u = \frac{V}{j\,d\,.\,\Sigma o}.$$

[For approximate results, j may be taken at $\frac{7}{8}$.]

5. *Columns.*

Total safe load,

$$P = f_c\,(A_c + n\,A_s) = f_c\,A\,(1 + (n-1)\,p).$$

Unit stresses, $f_c = \dfrac{P}{A\,(1 + (n-1)\,p)}$.

$$f_s = n\,f_c.$$

MINORITY REPORT.

To the Members of the American Society of Civil Engineers:

The undersigned, members of the Special Committee on Concrete and Reinforced Concrete, who did not indorse the majority report of the Committee as submitted to the American Society of Civil Engineers, beg leave to present their reasons for dissenting in some very essential points from the views of the majority of the members of the Committee.

1. Form of Report.—We do not consider the report as presented to be in proper shape for a document to be submitted to a society composed of engineers. As we understand the duty of your Committee to be "to investigate and report", we are of the opinion that the report should consist of a dissertation on our investigations, with the conclusions derived therefrom. Some chapters of this report are more in the shape of specifications or arbitrary rules, such as an engineer might prepare for the use of his subordinates. We consider the report as a whole to be hastily prepared, and neither carefully considered, nor systematically arranged.

2. Design.—The chapter on design is limited to a description of minor details of a particular kind of construction, generally, and sometimes improperly, used in reinforced concrete work of buildings, to the exclusion of other types; to which limitations a designer with more advanced ideas, who does not follow traditions, would naturally object. As the art of reinforced concrete construction is yet in its infancy, the tentative designs and methods adopted by the pioneers in their attempts to utilize a new material cannot be expected to be perfect, but more or less crude and unscientific, as has been the case in structures built of other materials, which, by the natural process of evolution, have gradually developed into more rational forms.

Had the report been prepared at the time your Committee was appointed, about four years ago, when this so-called progress report could have been made just as well as now, it would probably have been considered as giving a fair résumé of the state of the art at that time. The progress made and the experience gained in the last four years have not been given sufficient attention, and we consider the recommendations referring to design not broad enough and not up to date.

3. Working Stresses.—Scientific facts are not established by a vote. They should be deduced from experimental data and practical experience, and should appeal to reason and judgment. The working strains on concrete, as recommended, are arbitrarily assumed, without reference to any tests, or facts established by experience, and without giving any reason for departing from the practice established by years

of experience. In other words, the members of the Society are asked to believe in the superior judgment of the members of the Committee who are responsible for this report.

The properties of reinforced concrete are not yet as well established as those of nearly all other building materials, owing to the lack of conclusive tests. No recognized theory for computing the internal forces in reinforced concrete is in existence at the present time, and as the data which are now available have not yet been sufficiently confirmed by practical experience, they should be used with the utmost caution, so that if there be an error, it will be on the side of safety. The number of disasters in concrete structures has not diminished, but rather increased; a word of warning, therefore, would be appropriate at this time.

We are of the opinion that the acceptance of the report in its present shape would have a tendency to encourage questionable practice, which is not in harmony with the aims and objects of this Society.

It may be said that the Society disclaims all responsibility for statements and opinions expressed in any of its publications; nevertheless, the statements embodied in a report of a Special Committee appointed to report on a special subject by the American Society of Civil Engineers, if printed in its *Transactions*, will be quoted as being indorsed by that Society.

The members forming the minority of the Special Committee on Concrete and Reinforced Concrete are of the opinion that the majority report should be referred back to the Committee for revision.

Respectfully submitted,

JANUARY, 1909.

C. C. SCHNEIDER, *Chairman*,
EMIL SWENSSON,
J. E. GREINER,
OLAF HOFF.

DISCUSSION.

Mr. Watson.

WILBUR J. WATSON, M. AM. SOC. C. E. (by letter).—As stated in the Minority Report of this Committee, the Majority Report, or much of it, is based on arbitrary rules for designing and executing works in reinforced concrete.

Such a set of rules should be based on sound theory, which represents the best current practice, and should be thorough and complete. It appears to the writer that these three requirements have been violated in a few important particulars. In criticizing such a report as this, it should be borne in mind, however, that we are dealing with a material, or rather a combination of materials, the use of which in engineering construction is comparatively new, and the experimental data on which, while complete enough in some ways, are very meager in others. Consequently, much confusion exists among designers with regard to the rules which they use for designing and executing in this material.

Viewed as an effort to standardize such rules and practice, the report of the Committee is commendable. A report of this kind, by a committee of this Society, is generally accepted by the majority of engineers in the United States as representing the last word on any disputed point in engineering practice; and such reports should not contain any statements which cannot be backed up with conclusive experimental data or rigid mathematical analysis, and, in the writer's opinion, should contain no rules which must be considered as only tentative.

In regard to the corrosion of metal reinforcement embedded in concrete, the writer is somewhat disappointed to find no reference to the danger of corrosion of such steel when used as tensile reinforcement in beams with such high working stresses that serious tension cracks occur in the surrounding concrete.

The report states that concrete is sensitive to temperature changes, but does not state to just what extent. This information is needed, and would certainly be highly appreciated. It is the writer's notion, based upon observation of numerous concrete arch structures, that concrete is not sensitive to temperature, as compared with metallic structures. In fact, this statement of the report is partially contradicted in the next Section, where, under the caption "Fire-Proofing," the statement is made that "the low rate of heat conductivity of concrete is one reason of its value for fire-proofing."

The Section on shear and diagonal tension appears to the writer to be open to serious criticism. The action of shear members in providing against diagonal tension due to shear stresses is now pretty well understood. It is an established principle that, when the tensile stresses

Mr. Watson.

in a reinforced concrete beam exceed the allowed tensile stresses in the concrete, the metal reinforcement must be depended on to take care of all the tensile stresses without any reliance on the tensile strength of the concrete.

Precisely the same reasoning applies to diagonal tensile stresses, and, to be consistent, whenever the diagonal tensile stresses exceed the allowable tension in the concrete, metallic web members should have sufficient section to take care of the entire diagonal tension. It seems to the writer that it is unnecessary to give such indefinite methods as the Committee recommends for providing against shearing stresses. Such stresses are readily computed, and the necessary web members to provide for their resistance can be easily proportioned.

In Chapter VIII, Section 4, a working stress of 650 lb. for concrete in compression due to bending is recommended, but no reference is made as to whether such bending stress is to be computed by the "straight-line" or "parabolic" methods, which are both in use and with results varying by about 15 per cent.

The bonding stress of 80 lb. per sq. in. allowed in Section 6 is certainly much greater than is now used by the majority of conservative designers, who do not exceed 50 to 60 lb.

In Appendix B, the symbol P is used to denote the stresses in a single reinforcing member, while the same symbol is used to denote the total safe load on a column. The writer can see no reason for reducing the area of diagonal web members to seven-tenths of that required for vertical tension members, but recognizes the theoretical advantage of the diagonal web members.

The recognition of the desirability of continuous beam construction is to be commended, but the formulas are not those in common use, and the writer questions the wisdom of reducing the center moment below eight-tenths of that for a simple beam, as now commonly used.

On the other hand, no provision whatever is made for the increase in the shearing stresses due to continuity, and this appears to be a serious omission. The writer increases shears for continuous construction by 25% over that for simple beam construction.

This whole matter of continuity requires very careful consideration, and approximate methods of analysis should not be encouraged any more than is necessary. Most beams in reinforced concrete construction are T-beams, and the distribution of moments between the sections and the supports is far different from that in rectangular homogeneous beams, the moment at the supports being ordinarily decreased, and those in the center being increased, by reason of the T-section and the non-homogeneity of the material. However, very few designers provide sufficient reinforcement over the supports. In many cases, the writer has found that, with ordinary T-sections and

percentages of reinforcement, using a center moment for uniform loads equal to $\frac{W l}{10}$, slightly more than two-thirds as much reinforcement should be provided over the supports as is provided at the centers, and care should be taken not to overstress the concrete in compression on the under side of the beam next to the supports. Mr. Watson.

It might also be well to call attention to the fact that practically all columns in reinforced concrete construction are subject to more or less severe bending stresses, owing to the fact that, while the dead load is often symmetrical about the column, the live load seldom is, and the inequality in the distribution, in monolithic construction, of the live load always produces bending moments in the columns. It would seem to the writer, therefore, that it is never safe to omit the longitudinal reinforcing rods in columns of this class.

JOHN STEPHEN SEWELL, M. AM. SOC. C. E. (by letter).—The writer read this report when it was first published with a great deal of interest; but after studying it carefully, he is compelled to agree with the minority on practically every criticism offered. Most of the conclusions in the Majority Report are merely stated without demonstration of their correctness; the reasons for assumptions are not set forth. In a report of this kind, the members of the Society should be furnished with the data in order to enable them to judge of the correctness of the Committee's conclusions. Mr. Sewell.

If this Society were a body of laymen, and the Special Committee had been retained as experts to formulate rules governing the use of concrete and reinforced concrete, the form of the Majority Report might be accepted as correct. But these conditions do not obtain, and the members of the Society are entitled to the data and the reasoning which have led to the conclusions. It might even be helpful if the deduction of the somewhat complicated formulas were also included.

The writer strongly concurs in the recommendation of the minority that the report be referred back to the Committee for revision.

The following are some of the points that seem worthy of further attention:

DESTRUCTIVE AGENCIES.

Common Salt.—There is good reason for believing that when common salt, or sea salt, is mixed with the concrete, either to prevent freezing, or by reason of the use of sand or gravel from the seashore, or of sea water for mixing, steel embedded in the concrete will be corroded, especially if the concrete is in a damp location. It ought not to be difficult to determine the facts once for all.

Manure.—Fresh stable manure may have an acid reaction, but the writer has recently seen statements from expert chemists that manure

Mr. Sewell.

and urine are both alkaline. The chemists differed as to whether or not water that had filtered through manure- and urine-soaked ground would be deleterious to green concrete. If the members of the majority are in possession of positive evidence on these points, they should produce it.

Ensilage.—The writer has seen a well-hardened concrete silo badly pitted after one season's use for ensilage made of green cornstalks. As this use of concrete is likely to increase, such action should be investigated.

Oils.—The production of positive evidence as to the action of different oils, with a definite statement of the origin and composition of those having an injurious effect, would be of value. The statements in the report would not be of much assistance in practice.

Fire-proofing.

The treatment of concrete as a fire-resisting material by the Majority Report is inadequate. Igneous rock is not necessarily the best aggregate. It often spalls badly, and may be less desirable than limestone, notwithstanding the inevitable calcination of the latter. Broken bricks, broken terra cotta, gravel, furnace slag, and clinkers are all good aggregates for fire-resisting concrete—better probably than any kind of broken stone. Concrete not only suffers dehydration in a fire, but it often spalls in the same way as granite. The thicknesses recommended by the Majority Report for fire-resisting purposes are not sufficient to provide for spalling, and should be regarded as the minimum allowable under the most favorable circumstances. A higher standard should be set for more severe conditions. The desirability and possibility of freeing the fire-resisting covering—at least of columns—from structural stress, should receive consideration.

Reinforced Concrete.

It seems a little inconsistent to assume for the ratio, N, a value of 15 when designing the mid-section of a beam, and a value of 8 when computing the deflection of the same beam. If the methods, assumptions, and notations of the majority be adopted, it will be found that in designing rectangular beams when $f_s = 16\,000$, $f_c = 650$, and $N = 15$, $\frac{A}{b\,d} = 0.0077$. If the same beams be analyzed for stresses, it will be found that if $\frac{A}{b\,d} = 0.0077$, $f_s = 16\,000$, and $N = 8$, $f_c = 985$. There is little doubt that, for 1:2:4 concrete at an age of three months and upward, the true value of N is more nearly 8 than 15. 985 lb. per sq. in. seems high for f_c, yet no one doubts that the value of $\frac{A}{b\,d} = 0.0077$ is, in most cases, safe enough, even if it is intended to

allow f_s to reach a value of 16 000 lb. per sq. in. It seems to the writer both simpler and more accurate to determine the maximum allowable percentages of reinforcement from tests, and then to design the mid-section without reference to either N or f_c. Mr. Sewell.

The matter of web stresses and diagonal tension is very inadequately covered by the Majority Report. If it is unsafe to count on the tensile strength of the concrete to resist horizontal tension, it is equally unsafe to count on it for diagonal tension, or for shearing stresses in the web. The ultimate solution is to provide a complete system of inclined web members, rigidly attached to the horizontal reinforcement, and designed and spaced to take all the tensile web stresses; they should extend to the top of the beam and be designed so that their adhesion in the compressed part of the concrete will develop their full strength. The writer enunciated this principle, as a result of theoretical study and a few isolated experiments, more than five years ago. The recent experiments at the University of Illinois and the University of Wisconsin, in his judgment, have proven it correct. The recommendations of the Majority Report do not cover the ground, and do not establish a definite standard for the design of the web. Diagonal tension failures are the greatest danger to be feared in reinforced concrete work. Nearly every beam failure in practice has been of this kind. In the writer's judgment, the recommendations for the design of T-beams in the Majority Report would not suffice to avoid designs dangerously weak under the web stresses. Two layers of bars, one above the other, should not be allowed. A definite statement as to how the stress from the bars is to be transmitted through the rib and into the flange of a T-beam should be made a part of the report. The vague requirement for "effective bond" at the junction of rib and slab is not sufficient. A study of current practice will soon convince one that it is dangerously inadequate. A definite standard should be established. The effect of great depth, as compared with the span, in increasing the danger of failure under web stresses, should be treated at length.

The writer feels that recent experiments afford sufficient data for a much more positive stand than has been taken by the majority on this general subject of web stresses. Its position in some points is absolutely at variance with the results of some of these experiments. For instance, the tests reported in Bulletin No. 197, University of Wisconsin, show that loose stirrups are more efficient when inclined than when they are vertical. The requirement of the Majority Report that inclined stirrups must be rigidly attached, while vertical ones may be loose, seems to demand explanation. It is a little hard to see, also, why the Majority Report selects 75° as the angle of inclination. The tests in Bulletin No. 197 indicate that rigidly attached web members, whether inclined or vertical, are more efficient than loose

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ones; and that rigidly attached members are much more efficient when inclined at an angle of 45° than when vertical.

The many advantages of rigidly attached web members have seemed so obvious to the writer that he has often wondered why so many engineers have refused to admit them. It has occurred to him that the reluctance to commit one's self to a patented article may have influenced many purely professional engineers to a greater or less extent. It may be of interest to such men to know that the writer has had opinions from at least three competent patent attorneys and one patent expert, that the broad principle of attached web members is public property, and no valid patent, except on particular methods of applying it, can be obtained, or is in existence. At least three methods of applying it have been patented by three different inventors, and the writer has applied it successfully and economically with commercial bars, in a way that is not patented, and which, in the opinion of at least one competent patent attorney, cannot be patented. However, even if the broad principle were covered by patents, if it is the true solution, it should be adopted, and the inventor should get his reward. The writer asssumes that the Corliss valve gear was once patented, but if so, mechanical engineers did not deny their clients the advantages of Corliss engines until the expiration of the patents enabled any one to make and sell them.

Columns.

The writer has not had time to investigate for himself the majority's conclusions with reference to these members. But, in any event, these conclusions should be supported by a discussion of the data from which they were derived. If banded columns are to be used, it would seem that the bands should be designed, where the ends come together, so as to develop the strength of the material. It would also seem that a column should not be considered as hooped or banded unless the hooping or banding is circular.

Footings.

Current practice affords many instances of vicious and inadequate design of footings. This subject demands attention quite as much as any other.

There are many other points, such as permissible heights of reinforced concrete buildings; splicing of steel bars in superimposed columns; details of design at juncture of columns, beams, and girders, to resist secondary stresses; wind-bracing; resistance to earthquakes, shock, and vibration; design of outer walls and partitions in reinforced concrete; and design of structures as a whole, which demand attention, and on which the present state of the art affords, at least, a reasonable amount of information.

The writer feels that the Majority Report does not do justice to the able personnel of the Committee, and sincerely trusts that it may be referred back and carefully reconsidered. Mr. Sewell.

CLARENCE W. NOBLE, ASSOC. M. AM. SOC. C. E. (by letter).—The importance of the work of this Committee cannot be exaggerated. The Society has rarely, if ever, appointed a committee whose work has such a wide scope and is of such prime importance to the Engineering Profession. Mr. Noble.

The public expects that engineers will be governed in their work by two considerations: First, the consideration of safety, and, second, the consideration of economy.

In their anxiety to satisfy perfectly the first requirement, engineers must not overlook the fact that the second requirement must also be considered. If a design is safe, the addition of superfluous material does not increase its safety and does increase its cost. Skill in designing is the result of the application of adequate knowledge and of the use of material in such a manner as to insure economy as well as safety.

The consideration of economy must not be forgotten in the present discussion. Engineers now have a large amount of information regarding the stresses in a reinforced concrete beam when merely supported at the ends. They have a certain amount of information—and need considerably more—regarding the stresses in reinforced concrete beams when incorporated with others in floor panels in actual building construction. A series of carefully observed deflection tests taken on buildings which have been constructed and are in actual use would be of service.

At one time the writer had one of his designs submitted to such tests. He designed the reinforced concrete floors for a large denominational school. It happened that the architect did not belong to the same religious denomination as the school, and that he let the work at a cost plus per cent. basis to contractors who did not belong to this denomination. Later it was decided to extend the building, and contractors of the same religious belief as the school made a desperate attempt to discredit the architect by attacking the strength of the reinforced concrete construction.

To do this they tested the floor slabs in seven different places, the tests ranging from two to four times the safe load and extending over a width as great as the length of the panel. The loads were placed so as to avoid arching, and deflections were measured by a simple system of levers which multiplied the actual deflection by ten. The deflections of the adjacent beams were also noted, where they could be detected, and subtracted from the slab deflections. The results were entirely satisfactory from the writer's point of view, and the experience obtained was of considerable value.

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These slabs were supported on the exterior brick walls of the building on corbels, immediately above which there was built a reveal about 2½ in. deep. In some cases the interior supports consisted of reinforced concrete beams and in some cases of brick walls. All the bars were ordered long enough to lap well across the interior brick walls or concrete beams at the top of the slab, the intention being to build the concrete slabs across the walls and then continue the walls above them. The contractor, however, cut all the rods where they lapped across brick walls, and erected his brickwork in advance of the concrete, using a corbel and reveal on the interior walls as well as on the exterior walls. All the slabs were designed by the same bending-moment formula, regardless of location. Some of the slabs tested were supported on brick walls at both ends, some on brick walls at only one end, and some between two reinforced concrete beams.

While the deflections noted could not be actually predicted theoretically, owing to lack of knowledge of the properties of this concrete at the age at which these tests were made, the relations between the deflections of different slabs could easily be predicted.

In order to make a comparative study, all the deflections noted were reduced to terms of the deflections of a standard panel of a given depth. The assumptions made were as follows:

The deflection will vary directly as the load applied, directly as the cube of the span, inversely as the square of the effective depth, and inversely as the amount of reinforcement.

One of the test loads covered a panel only half as wide as it was long, and one of them, a short one, exhibited only one-fifth of the expected deflection. These two results are rejected in the following discussion. It was noted that the deflection from panels supported on two brick walls was considerably greater than that from those supported on two beams, although there seemed to be no difference between slabs having two beam supports and one wall support.

If it is assumed that the error arising from the use of the same denominator in the bending-moment formula is directly proportional in all cases to the variation in deflection, as exhibited in these standardized panels, the bending-moment formula should be as follows:

Supported at both ends, $\frac{w\,l^2}{8}$.

Both ends fixed, $\frac{w\,l^2}{14.3}$.

If consideration is given to the fact that the slab supported on brick walls at both ends had a certain amount of negative reaction, the bending-moment formula will be as follows:

Supported at both ends, $\frac{w\,l^2}{9}$.

Fixed at both ends, $\frac{w\,l^2}{16.1}$. Mr. Noble.

In the writer's opinion, the latter assumption most nearly represents the facts, although, in his practice, he uses $\frac{w\,l^2}{12}$ for slabs supported at one end and fixed at one end, and $\frac{w\,l^2}{16}$ is only used where an ample amount of negative reinforcement is provided above the beam.

It is quite desirable that this matter be thoroughly investigated experimentally, as there is probably at no other point a greater opportunity for economy than in the proper study of negative moment reactions.

In discussing floor slabs rodded in both directions, it must be remembered that the ratio between the loads transferred to the longer beams and the shorter beams is not fixed by any mathematical formula, as is the case where the slab is a plate equally able to transfer stress in all directions. On the contrary, the distribution is quite largely under the control of the designer, and depends on the ratio between the longitudinal and transverse reinforcement.

It has been the writer's custom to assume that the total load transferred to the longitudinal beams is the same as that transferred to the transverse beams. The slab can be considered as divided into four parts by its diagonals. Each of these four triangles will be equal in area, and will represent the equivalent of the load transferred to the beam which forms its base. The center of gravity of the load, however, is one-sixth of the span from the supporting beam, while if a similar diagram is drawn for a slab rodded in one direction only, the center of gravity is one-fourth of the span away from the beam, and the load is twice as great. For this reason the writer considers that only one-third as much reinforcement in each direction is needed to secure equivalent carrying capacity, as compared with the slab rodded in one direction only.

In order to assure these conditions, the long direction of the slab must be the more heavily reinforced. As the ratio between the longitudinal and transverse directions increases, the reinforcement in the longitudinal direction increases very rapidly and soon eliminates any economy resulting from the consideration of change in position of center of gravity of the load. This is quite in distinction with the result of variation in ratio between the two sides of the panel, when the rodding is done as suggested in the report of the Committee. In this case the allowable saving in reinforcement remains as great as ever, while the longitudinal bars in the extreme case must disappear.

In order to compare the proposed bending-moment formula with the result of the known tests, the writer has computed by this formula

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the safe load of various beams reported as being tested by A. N. Talbot, M. Am. Soc. C. E., in the now classic series of 1904, and this comparison is shown in Table 1. In making these comparisons only such beams as failed by tension in the steel are used.

TABLE 1.—COMPARISON OF BEAMS REPORTED IN BULLETIN No. 1, UNIVERSITY OF ILLINOIS, WITH PROPOSED FORMULA.

Formula: $M = f_s A j d. \quad j = \frac{7}{8}$.

Beam No.	Area of metal.	Kind of steel.	Safe M from formula, in foot-pounds.	Ultimate M actual test, in foot-pounds.	Factor of safety.
21	0.59	Mild.....	8 160	21 850	2.68
19	0.59	Mild.....	8 160	24 600	3.01
16	0.75	Mild.....	10 500	26 100	2.48
17	0.75	Mild.....	10 500	25 200	2.40
15	1.20	Mild.....	16 800	38 800	2.31
10	1.20	Mild.....	16 800	36 500	2.17
4	2.00	Mild.....	28 000	51 400	1.83
14	1.60	Mild.....	22 400	42 200	1.88
5	1.20	Mild.....	16 800	33 100	1.97
13	1.40	High carbon....	19 600	67 000	3.42
20	1.00	High carbon....	14 000	49 600	3.55
2	1.00	High carbon....	14 000	47 300	3.38
7	0.60	High carbon....	8 400	33 600	4.00
3	0.60	High carbon....	8 400	31 200	3.72

The average factor of safety of the nine beams reinforced with mild steel is 2.3; the greatest variation from this average is 31 per cent. The average factor of safety of the five beams reinforced with high-carbon steel is 3.61, and the greatest variation from the average is 11 per cent. The writer was much surprised to note that the use of high-carbon steel was evidently considered in the proposed specifications as of somewhat doubtful value. He was also surprised to note the low factor of safety, as shown by the formula, when compared with these tests. It would be interesting if the Committee would explain how these conclusions were reached. Surely the proposed specifications should be amended so that high-carbon steel can be used at a working stress somewhat near the same fraction of its elastic limit as is used for mild or medium steel.

The writer must confess to a great respect for re-rolled rails. This material is, everything considered, the cheapest reinforcing material on the market. At the time steel rails first began to be used for concrete reinforcement, the mills manufacturing this product did not fully understand the requirements of this trade. They were accustomed, therefore, to roll rails of all sizes indiscriminately into reinforcing bars. The same shipment would contain material from rails of 90-lb. weight containing from 48 to 58% carbon, and rails of 50-lb. weight containing from 35 to 45% carbon. As a result, the

product had a wide variation in elastic limit, and has come to be regarded among engineers as unreliable and unsatisfactory. Mr. Noble.

The writer has recently been using considerable quantities of re-rolled material from a mill which makes a practice of selecting only the smaller rails for concrete reinforcement, and of testing carefully all the product. On a series of thirty-four tests, made on a recent shipment, the average elastic limit was 63 000 lb. per sq. in., the average ultimate strength, 109 300 lb. per sq. in., and the average elongation, 14.07 per cent. On a series of 142 tests of this material, taken just as they ran, the highest elastic limit found was 94 000 lb. and the lowest, 45 200 lb. The highest ultimate strength was 137 900 lb. and the lowest, 66 500 lb. The number of bars having an elastic limit below 5 000 lb. was only 8, and the number of bars having an elastic limit above 75 000 lb. was 10. Only 4 had an elongation below 10%, and 27 had an elongation above 20 per cent. Bending tests showed that the majority of these bars would bend around twice their own diameter provided the bend was made slowly, but if it was made on an anvil, by hammering into position, a small percentage would break before reaching this stage.

It is quite important that structural steel be ductile, as it is necessary to punch and shear it during fabrication, and also because of the vibration to which it is subjected. Neither of these reasons apply to reinforcing bars. They are not punched or sheared, are bent more economically hot than cold, and are not subject to any great amount of vibration. For this reason the writer prefers high elastic limit to ductility, particularly as high elastic limit is so productive of economy.

In the writer's opinion, the practice proposed by the Committee has two faults which, in a measure, compensate each other. In the first place, the factor of safety resulting from the proposed formula for moment of resistance is too low. In the second place, providing a sufficient negative reinforcement is used, the factor of safety resulting from the proposed bending-moment formula is too high. The result, therefore, is somewhat near the truth for standard conditions, but is apt to vary if applied rigidly to cases which differ from ordinary practice.

S. Bent Russell, M. Am. Soc. C. E. (by letter).—After careful consideration of the reports made by the Committee, in the light of papers and discussions published previously in the *Transactions* of the Society, the writer is of the opinion that some of the points made by the Minority Report, in the second and third counts, are not without good foundation. Mr. Russell.

On examination, the objection of the minority in its second count seems to be on the question of the scope of the inquiry. The report has gone beyond that field of construction which is subject to

Mr. Noble.

the safe load of various beams reported as being tested by A. N. Talbot, M. Am. Soc. C. E., in the now classic series of 1904, and this comparison is shown in Table 1. In making these comparisons only such beams as failed by tension in the steel are used.

TABLE 1.—COMPARISON OF BEAMS REPORTED IN BULLETIN NO. 1, UNIVERSITY OF ILLINOIS, WITH PROPOSED FORMULA.

Formula: $M = f_s A j d. \quad j = \frac{7}{8}$.

Beam No.	Area of metal.	Kind of steel.	Safe M from formula, in foot-pounds.	Ultimate M actual test, in foot-pounds.	Factor of safety.
21	0.59	Mild.....	8 160	21 850	2.68
19	0.59	Mild.....	8 160	24 600	3.01
16	0.75	Mild.....	10 500	26 100	2.48
17	0.75	Mild.....	10 500	25 200	2.40
15	1.20	Mild.....	16 800	38 800	2.31
10	1.20	Mild.....	16 800	36 500	2.17
4	2.00	Mild.....	28 000	51 400	1.83
14	1.60	Mild.....	22 400	42 200	1.88
5	1.20	Mild.....	16 800	33 100	1.97
13	1.40	High carbon....	19 600	67 000	3.42
20	1.00	High carbon....	14 000	49 600	3.55
2	1.00	High carbon....	14 000	47 300	3.38
7	0.60	High carbon....	8 400	33 600	4.00
3	0.60	High carbon....	8 400	31 200	3.72

The average factor of safety of the nine beams reinforced with mild steel is 2.3; the greatest variation from this average is 31 per cent. The average factor of safety of the five beams reinforced with high-carbon steel is 3.61, and the greatest variation from the average is 11 per cent. The writer was much surprised to note that the use of high-carbon steel was evidently considered in the proposed specifications as of somewhat doubtful value. He was also surprised to note the low factor of safety, as shown by the formula, when compared with these tests. It would be interesting if the Committee would explain how these conclusions were reached. Surely the proposed specifications should be amended so that high-carbon steel can be used at a working stress somewhat near the same fraction of its elastic limit as is used for mild or medium steel.

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On examination, the objection of the minority in its second count seems to be on the question of the scope of the inquiry. The report has gone beyond that field of construction which is subject to

Mr. Russell.

practical experiments, and has chosen certain forms of construction presumably in common use, and it may be thought, therefore, that the Committee has discriminated against other forms of construction.

In the writer's judgment, the report could be made stronger and more effective by limiting the field so as to confine the recommendations to matters which can be determined by practical tests. To this should be added, however, a plea for caution in applying the recommendations outside of their proper scope. In other words, the Committee should make recommendations for the design and for unit stresses to apply to rectangular and T-beams supported at the ends and for columns in axial compression. Sufficient experimental investigation has been made to warrant such recommendations, and they would always be subject to proof or correction by subsequent tests.

In the field of continuous beams and other complex structures, the Committee should avoid making definite recommendations; but, in order to prevent wrong uses of its specifications, it should certainly call attention to usual errors in the design of such structures, such as the omission of top reinforcement over the supports of continuous beams, lack of provision for eccentric loads, temperature stresses, etc.

The report could be made to conform to the lines herein suggested without much alteration in form, and it would then offer much less ground for criticism in reference to arbitrary recommendations.

It may be added that the proposed changes would also remove much of the objection shown in the third count of the minority, as the working stresses recommended would be in the field of practical tests.

Mr. Worcester.

J. R. WORCESTER, M. AM. SOC. C. E.—The reasons of the minority of the Committee for not signing the report of the majority are, in substance:

1.—That the report is hastily prepared, without careful consideration, and not in proper form.

2.—That it refers to one type of construction, without breadth enough to be applicable to designs which the future may develop.

3.—That the working stresses recommended are not supported by the evidence of tests.

4.—That the acceptance of the report would tend to encourage questionable practice.

The duties of a special committee of this Society are not definitely laid down, but are controlled by the judgment of the committee itself. This Committee was directed to "take up" the question of concrete and steel concrete. The reason for its appointment was the realization by the Society that there was grave danger in the rapidly increasing use of these materials without adequate scientific investigation and determination of properties, and the general diffusion of knowledge based on such studies. At the time, the reinforced concrete trade was controlled mainly by companies claiming proprietary rights and special

knowledge of properties, and the public was dependent on the catalogues and contracting agents for information as to what dimensions and details to use. At the same time, many contractors, in absolute ignorance of the science, were claiming that they could handle the materials as well as any one. City authorities were being beset with applications for building permits based on material not mentioned in their laws, and were in grave perplexity as to what restrictions to impose. The cement manufacturers were as much troubled as consumers over the situation, realizing the danger to their trade from reckless use of their product. Under these circumstances, the Committee's understanding of its duties was to make sufficient study of the properties of concrete and reinforced concrete to enable it to lay down general rules for proportioning, which would result in safe construction. While the Committee has never formally adopted this wording as the objective at which it aimed, it seems to the speaker to have been so admitted, tacitly, by all members. Nearly five years elapsed from the time the Committee was appointed until this progress report was presented, and it can hardly be urged that undue haste was exhibited in giving something to the Society. To be sure, not all of the time was spent in the preparation of the report, but a full year was spent in this part of the work, and the remainder of the time in the accumulation of information. During the five years that the Committee has been in existence, many of the reasons for its appointment have disappeared, through the untiring efforts of many investigators and the gradual dissemination of knowledge. It is probable that if its report were delayed long enough, it might be saved the trouble of making any report at all, as it would not be needed. There still remains, however, a considerable divergence of views on many of the points treated in the report, as is evident from criticisms which have appeared. As to the form of the report not being correct, the majority would gladly see it improved, but considers form subordinate to substance. Mr. Worcester.

In answer to the criticism of the minority that the report treats of one style of construction only, it may be said that it deals with every kind of stress to which concrete is subjected, and includes all ordinary conditions of proportioning and handling. If, in the future, new methods of designing introduce conditions not covered by these rules, they may be studied as developed. It is undoubtedly true that some special designs, involving indeterminacy of analysis by the ordinary rules of mechanics, are already extensively used, and these may well deserve experimental study. The existence of such designs, however, should not delay a report on other types which are in universal use and have been carefully investigated.

As to the tests on which the report is based, the majority of the Committee did not consider it necessary or useful to submit with its report the voluminous data at its disposal. Many of these data are

Mr. Worcester.

already in print, and the remainder are likely to be published sooner or later. If they were all reprinted at this time, they would be of little use except to an investigator with unlimited time at his disposal, who wished to check the deductions made by the Committee. Moreover, the recommendations, particularly those contained in Chapters III, IV, V, and VI, are largely the outgrowth of the experience in actual construction of members of the Committee and many others, rather than any definite series of tests. The majority feels safe in asserting that no recommendation is made in the report which is not based on good grounds. Any questions as to specific points in the report will be gladly welcomed by the Committee, and, in answering such questions, the majority may perhaps set at rest any doubts as to the sufficiency of evidence on which the recommendations are based.

The last charge of the minority, namely, that the acceptance of the report would tend to encourage questionable practice, is the most serious, and, if substantiated, would be sufficient reason for the refusal of the Society to give credence to the report. It is difficult to meet a sweeping assertion of this kind without specific mention of the particular features on which it is based. The fact that, since its presentation, this report has been freely criticized as being too conservative, is an indication that much of the practice of the day is more radical than that recommended. It may be, as asserted, that "the number of disasters in concrete structures has not diminished, but rather increased," but the minority cannot mean to imply that any disaster has ever occurred in construction executed in accordance with the recommendations of the report; and, it may be stated further, without fear of contradiction, that no failure has occurred in which principles laid down in the report have not been distinctly violated. In recommending unit stresses, the Committee has advised safety factors higher than those allowed in steel construction, and everywhere has correlated them to the quality of the concrete used, so as to insure the preservation of the desired factor. In the principles of design, no refined mathematical theories are advanced, but assumptions are adopted, which have been amply proved to be safe, and the Committee has repeatedly called attention to features wherein more experimentation may lead later to greater economy than can safely be recommended at the present time. The majority members of the Committee have felt their full responsibility to the Society and to the Profession, and have no interest in advocating any questionable practice. They rest assured that this charge will not be taken seriously by any engineer who makes a study of the report.

In conclusion, the speaker wishes to submit that, in view of the facts, the statements of the minority are not substantiated. Believing that concrete and reinforced concrete have become established among the most useful materials of construction, and are being used in every

branch of engineering more extensively every day; that rules for the use of these materials have been adopted already by many nations, as well as municipal governments; that other technical societies are adopting rules based on no more complete data than are available to this Society; and that construction will not await absolutely complete scientific demonstration of a material before adopting it, are we not justified in advancing, in a progress report, tentative conclusions? Mr. Worcester.

W. J. DOUGLAS, M. AM. SOC. C. E. (by letter).—The writer has read the Minority and Majority Reports of this Committee with much interest, and desires to offer some criticism. He is of the opinion that the report of the majority should be adopted, after some changes have been made in it and after additional matter has been inserted. Mr. Douglas.

In regard to the Majority Report, the writer is of the opinion that it would have been strengthened very much if the data on which the permissible unit stresses were determined had been stated, and, while approving in a general way of the unit stresses adopted, he thinks that it would be a serious mistake to adopt the report without such additional information as would permit the Society to use the suggested unit stresses with a better understanding of the basis on which they were determined. One should bear in mind, in using the stresses recommended, that they are based on first-class work, and are high, unless the work is to be executed under more competent supervision than reinforced concrete work, in general, receives. Most building departments, improperly, do not require competent certificates of cement and steel tests, and are not equipped with competent inspectors, and such departments would not be justified in using the high stresses allowed for concrete in the Majority Report. The writer believes that the report should be referred back for revision, in order to perfect its usefulness, but he is strongly of the opinion that it is basically excellent, and needs only slight revision, in order that it may serve more fully the purposes for which it is intended.

In considering the Minority Report, it might be well to note that the art of reinforced concrete construction is severely criticized, such terms as, "infancy," "pioneers," "crude materials," and "unscientific," being used, and by these expressions and the further statement that: "The number of disasters in concrete structures has not diminished, but rather increased; a word of warning, therefore, would be appropriate at this time," it is shown clearly that the minority is somewhat prejudiced against this type of construction.

In the Majority Report, it is stated: "Plain concrete or massive concrete is well adapted for structural forms in which the principal stresses are compressive." This includes "* * * in many cases, arches." It is thought that this latter statement is somewhat mis-

Mr. Douglas.

leading, in that steel reinforcement in arches of ordinary rise and correct form is not economical, except where the probable deformation of the foundations makes reinforcement advisable.

In Chapter II, Section 2, it is thought that Paragraphs 1, 2, 3, 5, and 7, would better be omitted, as they apply to all classes of structural design, and not specifically to concrete work.

The writer thinks that it would be advisable to add the following to Paragraph 4 of this chapter: The design should show typical laps and bends and typical slabs, beams, girders, and columns, properly dimensioned, to insure the packing of all steel members. All beams which have been figured upon, in the design, as T-beams, should be indicated on the plans, and, if a sufficient number of shear bars have been incorporated in the design to take the entire horizontal shear between the stems and the flanges of the T-beams, this also should be noted on the design.

In regard to load tests, Chapter II, Section 3, it is believed that the test loads should not exceed twice the live load, as otherwise the elastic limit of the steel reinforcement may be exceeded.

The addition of the following is suggested to this paragraph: Tests should be made on unfinished floors (that is on floors upon which the finished surfaces of strip and strip filling have not been placed).

Tests should be made with materials, such as water or sand (not in bags), which do not arch themselves. Concentrated test loads should not be used except to determine the efficiency of the floor system to withstand concentrated loads.

In Chapter III, Section 2, it is stated:

"Mortars composed of one part Portland cement and three parts fine aggregate, by weight, when made into briquettes should show a tensile strength of at least 70% of the strength of 1:3 mortar of the same consistency made with the same cement and standard Ottawa sand."

This is an excellent requirement, and one which is seldom incorporated in specifications.

In Chapter III, Section 4, it is stated that the reinforcing steel should fulfill the requirements for structural steel. This is thought to be an unreasonable requirement, except where the reinforcing steel consists of built-up structural members. Such reinforcements, of course, would receive the same injurious treatment (such as punching) that structural steel members receive. Nor does it appear to be advantageous to exclude high-carbon steel, if such steel gives proper bending tests. There can be no reasonable doubt that, for reinforcing concrete, steel of a high elastic limit has material advantages over steel of a low elastic limit.

In specifying the quality of reinforcing steel which is to be bent only, and then rarely, at angles of less than 90°, it should be sufficient

Mr. Douglas.

to stipulate the minimum ultimate strength, elastic limit, and bending quality. Cold bending gives the best test to determine the suitability of steel reinforcement, and it is easily made in the field. The writer, therefore, suggests the following:

Reinforcing steel should have an ultimate strength of not less than 60 000 lb. per sq. in., and an elastic limit of at least 50% of its ultimate strength, and, for sizes $\frac{3}{4}$ in. or less in diameter, should bend cold 180° around its own diameter without fracture. For sizes greater than $\frac{3}{4}$ in. in diameter, it should bend 180° around twice its own diameter without fracture, except in cases where all bends necessary in the construction are to be made hot, in which case the steel should bend cold 90° around its own diameter without fracture. In the case of flat bars or structural shapes, the reinforcing steel should be bent cold around its own thickness, or twice its thickness, respectively. Tests on the ultimate strength, elastic limit, and bending quality of the steel should be made in the form (as to cross-section) in which it is to be used. Steel reinforcement made from re-rolled rails should not be used. If deformed bars are to be used, only the net section—the section exclusive of all projections—of the areas of such bars should be considered effective.

Bars showing fractures at bends should not be used.

All reinforcing steel should be free from paint, dirt, oil, grease, scale, and rust flakes, before being encased in concrete.

It is thought to be impracticable and unnecessary to stipulate that steel should be free from rust, and the writer thinks that it would be better to substitute the term, rust flakes, instead of rust.

In Chapter IV, Section 3*a*, it is stated that special care should be exercised to prevent the formation of laitance. The writer knows of no way of doing this. The only practical way to prevent the formation of such a large quantity of laitance as will do material harm to the foundations is to lay the concrete as continuous work, and as quickly as possible.

In Chapter V, the Committee does not go very deeply into the question of forms, and as this is of vital moment, the elaboration of this article, to include the following requirements, is suggested:

All forms—and supports for forms—should be strong and stiff, true, out of wind, practically unyielding, and sufficiently tight to hold the liquid mortar without leakage; and their interior dimensions should conform to the dimensions of the concrete sections shown on the approved plans. The lagging—when lumber lagging is used—should be of a good quality of lumber, dressed to matched sizes. This lumber should be dressed on one face and two edges, except when the exposed surface is to be plastered, in which case the dressing should be omitted.

Forms which, at any time, become twisted or warped, should be removed or correctly adjusted before concrete is placed within them.

Mr. Douglas.

All forms—and supports for forms—should be designed to carry safely the weight of the forms, the liquid concrete being assumed to weigh 150 lb. per cu. ft., and a live load, in addition thereto, over the floor-slab area, of 25 lb. per sq. ft. They should also be designed to withstand safely a wind pressure of 20 lb. per sq. ft. upon all surfaces exposed to the wind.

The forms for beams, girders, and lintels should be designed so that at least one side of each beam and girder and lintel may be removed without disturbing the bottom portion of the forms and its supports. All posts supporting forms for slabs, beams, and girders should rest upon wedges which may be loosened and removed without producing undue strains in the floor system.

Posts which are to be supported by the ground should have their loads distributed by spread footings, so that there may be no appreciable settlement of the posts due to settlement of their foundations.

Columns with or without bands or hoops and with vertical reinforcement, in which there is a clear space of 5 in., or more, between the vertical units of rods, bars, or shapes, and where the vertical bars are wired so that the clear space between wires in the same horizontal plane is not less than 8 in., and where the vertical reinforcement has no attached web or shear members, should be poured—filled with concrete—for heights not exceeding 20 ft., without leaving one side of the column form open, except that, at the bottom of all columns, one side of each column form should remain open until about an hour before the filling of the column form is begun.

All columns not conforming to the above specifications should have one side of each column form open until concreting is begun in it, and its open side should be lagged up as the concreting progresses, but in no case should the top of this lagging be more than 3 ft. above the level of the column concrete.

In the case of columns of circular cross-section, the phrase, "one side open," should be taken to mean that, in horizontal cross-sections, at least one-sixth of the perimeter of the circular form should be open.

All shavings, chips, sawdust, and other foreign matter should be removed from within the forms before concrete is placed therein.

While agreeing with the report as to the time of the removal of forms, the writer thinks a general minimum might be stated as a guide, and suggests the following:

The season favorable to the laying of concrete (when the temperature does not fall lower than a few degrees below freezing) should be lengthened in the lower latitudes and shortened in the higher latitudes, and the unfavorable season should be correspondingly shortened or lengthened.

Mr. Douglas.

Forms should not be removed in less time than is specified below:

Slabs and lintel forms, April 1st, to November 1st:

Spans less than 3 ft.	3 days.
More than 3 ft. and less than 5 ft.	4 days.
More than 5 ft. and less than 8 ft.	5 days.
More than 8 ft. and less than 13 ft.	6 days.
More than 13 ft.	7 days.
Slab and lintel forms, November 1st to April 1st.	12 days.
Supports for beams and girders, April 1st to November 1st.	15 days.
Supports for beams and girders, November 1st to April 1st.	21 days.
Columns and wall forms, April 1st to November 1st.	3 days.
Columns and wall forms, November 1st to April 1st.	12 days.

The above times of removal should be estimated from the time the concrete is completely placed in any section of the work.

Before undertaking to remove any form work, the superintendent or foreman in charge of the work should satisfy himself as to the condition of the concrete, and if it does not ring when struck by a hammer, he should call the attention of the engineer to the same and delay the work of removing the forms until the hammer test is satisfactory.

Before removing the temporary supports of the beams and girders, at least one side of the adjacent column forms and one side of each beam and girder form should be removed in order to expose the concrete to view, so as to give evidence of the soundness and hardness of the concrete.

When forms are being removed, there should be no load upon the portion of concrete affected in excess of one-sixth of the live load for which the portion affected was designed, unless temporary posts are left in to take care of such load.

Reinforced concrete floors—this does not apply to reinforced concrete slabs laid upon steel beams and girders—should be supported by continuous lines of temporary vertical posts—sufficient to carry the floor weights—extending down to the basement or to a floor which is strong enough to bear the upper floor loads. These props should be removed at such times as the upper floors have hardened sufficiently to carry safely their own load and such floors above them as are unable to carry their own weight safely.

The times of removal given hereinbefore are based on the assumption that the upper floors are sufficiently strong to carry their own weights, or that continuous lines of vertical posts have been provided to take care of the upper floor loads.

In Chapter VI, Section 1, it is stated that "in columns, large bars

Mr. Douglas.

should be properly butted and spliced." This paragraph also states that "at foundations, bearing plates should be provided for large bars or structural forms." It is thought that it should be specifically stated that the ends of large bars should be milled, and that, instead of requiring bearing plates, these requirements should read, bearing plates or bars. Wide plates are objectionable where columns rest upon spread reinforced concrete footings, on account of making a horizontal shearing plane of weakness.

In Chapter VI, Section 2, it is stated:

"Girders should never be constructed over freshly formed columns without permitting a period of at least two hours to elapse, thus providing for settlement or shrinkage in the columns."

It is an open question whether or not it is necessary to pour a column in advance of the girder which it supports. The writer recently poured columns 42 ft. in length, and immediately on their completion, the cross-girders were poured, and the work is entirely satisfactory. About the only condition which would make any difference would occur if the girder set up more rapidly than the column, or in the case of a very large column, as in a lower story of a building, supporting a small beam, in which case the column concrete in setting might rob the girder concrete to the detriment of the latter. And it further appears that if it were necessary to have a time interval between the pouring of the columns and the girders which they support, two hours would not be sufficient.

In Chapter VII, Section 1, it is stated that: "Massive concrete is also a suitable material for arches of moderate span where the conditions as to foundations are favorable." This sentence would conform more closely to the accepted practice if the words, "of moderate span," were stricken out.

In Chapter VII, Section 3*b*, it is thought that all beams and slabs estimated as continuous or restrained beams should have their span lengths taken from center to center of supports, and that the load upon the beams and slabs should be taken on the clear spans. In simple beams and slabs the span length should be taken from center to center of supports, but should not exceed the span length plus the necessary bearing length. The span length plus the depth may result in an absurd length of beam and of slab.

In Section 4, it is thought that a better shaped T-section would result if the additional restriction were made that the width of the flange should not exceed five times the width of the stem.

It is thought that the following additional restrictions might properly be added: If shear bars are necessary to take care of the shearing stresses, they should extend at least to the center of the slab, and there should be sufficient steel in the slab transverse to the

beam or girder to take care of the negative bending moment in the slab, and said steel should be located so as to take care of the negative bending moment effectively. This restriction is important, because, if the negative bending moment is not taken care of, the beam may crack and reduce the effective width of the flange below that calculated in the design. Mr. Douglas.

The writer would suggest the following at the end of Chapter VII, Section 6:

When the bottom reinforcing bars are bent up at the ends of a beam, for shear and negative bending moment, care should be taken that the bottom bends are not made so close to the middle of the beam that there is insufficient horizontal reinforcement near the supports to take care of the positive moment.

In Chapter VII, Section 7, it is stated in the last paragraph that:

"The lateral spacing of parallel bars should not be less than two and one-half diameters, center to center, nor should the distance from the side of the beam to the center of the nearest bar be less than two diameters. The clear spacing between two layers of bars should not be less than ½ in."

The writer is of the opinion that the allowed lateral spacing of bars, in systems which provide proper spacers or separators, may be less than in those which do not have such spacers or separators, because, in the latter systems, the horizontal bars become more closely bunched together than the drawings indicate. In many cases they are found in contact for a considerable portion of their lengths.

If there are two layers of bars, those of the upper layer should be spaced farther apart than those of the lower layer. If there is a third layer, the spaces between the bars of the uppermost layer should be still greater than those between the bars of the second layer.

It is thought that the introduction of the expression, "diagonal tension," in Chapter VII, Section 8, may be somewhat misleading to many, and, as the report of the Committee does not include any formula for diagonal tension, but only for shear, it would be better not only to omit this expression, but to state clearly that if the vertical and horizontal shear are taken care of properly, the diagonal tension need not be considered.

On page 450 general rules for shear reinforcement are given. If the Committee means that it is good practice to allow a value of 120 lb. per sq. in. for shearing stress in concrete, it would be of interest to the Society to know the experimental data on which this is based. These general rules might be better divided into:

a. For beams having horizontal bars only;

b. For beams in which the shear is partly taken care of by bent-up rods, shear bars, or both.

Mr. Douglas.

Under Section 8 it would be advisable to prohibit the eccentric attachment of shear bars to round rods, because this eccentric attachment tends to cause these bars to rotate, and thereby decreases the available adhesion.

In Chapter VII, Section 9, it is stated: "It is recommended that the ratio of the unsupported length of a column to its least width be limited to 15." The writer does not believe that the ratio of the unsupported length of the column to its least width should be limited to 15. This seems to be an unnecessary requirement to put on the designer who will occasionally need to support light loads, such as roof loads, by the use of long columns, and if he reduces the working stresses in the concrete and steel, there is no apparent reason why he should not use a column twenty-five times its least width, if this column is properly reinforced. It is suggested that, on the basis of the unit stresses allowed, the ratio should be limited to 15, and that if a higher ratio of length to least width is used, the allowable unit stress in the concrete steel be reduced on the basis of the following formula or one similar thereto:

$$f_c' = f_c'' + 450 - \frac{30\,L}{D}.$$

f_c' being the unit stress (from 450 to 600) allowed in the column concrete for columns in which $\frac{L}{D}$ exceeds 15, and f_c'' being the unit stress allowed in the column concrete for columns in which $\frac{L}{D}$ is 15 or less than 15.

Under Section 9, on columns, it would be advisable to state that if the structural steel portion of the column is in any form which tends to divide the column in its horizontal cross-section into independent parts, the concrete should be regarded as fire-proofing only. For instance, where the steel is in the form of an I or star shape, and in which the reinforcing steel extends to within a few inches of the outside of the concrete, it would not be good practice to assume that the concrete would carry any portion of the column load. By hooping the concrete it might be made to carry its portion of the load, but such design would be manifestly uneconomical.

On page 451, working stresses in the concrete for various types of columns are recommended.

Is there any reason for allowing as high stresses in the concrete of a structural reinforced column as in that of a hooped column? One cannot regard a column built of structural shapes as a hooped column. Hooping must be circular, not square. Bands built in the form of squares would certainly strengthen the column, but it is believed to a much less degree than if the bands were circular.

Mr. Douglas.

It is thought that the allowable unit stresses in *d* should be intermediate between those of *a* and *b*.

In connection with the working stresses in columns, the writer thinks that in *a* and *d*, a limit to the percentage of steel reinforcement should be stated. A high percentage of steel, say 9% or 10%, would cut up the column concrete so that it would not be safe to stress it as highly as if a lower percentage of steel had been used.

The writer suggests the consideration of a specification in general in accordance with the following:

For the sake of stating a general rule, 600 lb. per sq. in. is assumed as good practice for 5% reinforcement.

In columns with longitudinal reinforcement only, the following unit compression per square inch will be allowed:

For columns not exceeding 5%, reinforcement 600 lb.
For columns exceeding 5% but not exceeding 6%, 575 lb.
For columns exceeding 6% but not exceeding 7%, 550 lb.
For columns exceeding 7% but not exceeding 8%, 525 lb.
For columns exceeding 8% but not exceeding 10%, 500 lb.

Chapter VIII, Section 2, states:

"When compression is applied to a surface of concrete larger than the loaded area, a stress of 32.5% of the compressive strength at 28 days, or 650 lb. per sq. in. on the above-described concrete, may be allowed. This pressure is probably unnecessarily low when the ratio of the stressed area to the whole area of the concrete is much below unity, but is recommended for general use rather than a variable unit based upon this ratio."

It appears that the allowable compression for the load area is stated too indefinitely, and that the pressure allowed is low.

The rule which specifies the allowable unit pressure "when compression is applied to a surface of concrete larger than the loaded area," gives the designer too much leeway. For instance, he might take advantage of the higher allowable compression, when the surface of the concrete was only 5 or 10% greater than the loaded area. On the other hand, if the area of the concrete surface was ten times the loaded area, the allowable unit stress is low.

The writer would suggest the following, which he believes is based on tests, and on sound logic, independent of recorded tests:

When a compression of three times the loaded area and less than ten times the loaded area, is applied to a surface of concrete, a compressive stress of 800 lb. per sq. in. of the loaded area will be permitted. When a compression of more than ten times the loaded area is applied to a surface of concrete, a compressive stress of 1 000 lb. per sq. in. of the loaded area will be permitted.

In Chapter VIII, Section 6, it is noted that the adhesion allowed for drawn wire is less than that allowed for bars. The reason for this

Mr. Douglas.

is not apparent, because, while it is of uniform cross-section, it is far from straight.

The writer would suggest the desirability of publishing, as part of this report, the correct method for applying the shear formula given on page 464, as the average engineer engaged in reinforced concrete work finds the application of this formula somewhat difficult.

For simplification, the writer advocates that the shear in restrained and continuous beams be assumed as equal to that of simple beams.

The following additional data are believed to be worthy of consideration in the formulation of general rules for the design and construction of reinforced concrete work, and might properly be considered by the Committee, should the report be referred back for revision:

After the concreting of a slab, beam, or girder is begun, it should be carried on continuously until the full depth of the slab, beam, or girder is reached. By "continuously" is meant that adjacent layers of concrete should be placed within 90 min. of each other.

When T-beams or T-girders have been figured on in the design—which should be indicated on the plans—the slab should be laid continuously with the beam or girder concrete.

Beams and girders which are calculated as simple beams, but are built continuously over supports, should be reinforced at the supports, in the upper portion of the beam or girder, with steel having a cross-section equal to one-quarter of the cross-section of the steel necessary in the bottom of the beams and girders at their middle. This upper reinforcement is introduced in order to prevent cracking, and should extend a sufficient distance beyond the center of the supports in order to develop adhesion equal to the strength of the upper bars, on the basis of the allowable unit stresses given.

Reinforcing steel for slabs, calculated to carry stresses, should not be spaced farther apart than two and one-half times the depth of the slabs, for slabs not exceeding 4 in. in depth, and not farther than twice the depth of the slab, for slabs exceeding 4 in. in depth.

Main tension members, for slabs, beams, and girders, should not be spliced except over supports.

The depth of beams and girders—measured below the bottom of the slabs, in the case of T-beams—should not exceed three times the width of the beam.

The steel of beams and girders should be designed so as to tie the building together securely at story levels.

Notches in the walls to receive the bearing ends of slabs should not be permitted, unless the height of the notches be three or more times their depth—horizontal dimensions being measured at right angles to the wall. Where notches are used at the bearing ends of slabs, the

reinforcing steel should extend into the notches to at least one-half their depth. Mr. Douglas.

Floor slabs designed to carry heavy concentrated loads should be provided with transverse as well as longitudinal reinforcement, so as to distribute the concentrated loads.

In combination concrete and tile construction, the dead and entire live loads should be assumed to be carried by the concrete beams.

At floor levels, where all the column reinforcing rods are designed to abut, additional steel rods should be used to tie the columns in consecutive stories. This steel should have an area of at least two-thirds of 1% of the effective area of the column, and should be of such length that the adhesion will develop the strength of the bonding steel.

Longitudinal steel column reinforcement should not be butted or spliced farther than 2 ft. from floor levels.

Where steel reinforcement is not required in a column for compression, an amount of steel at least equal to two-thirds of 1% of the cross-sectional area of the column should be used.

C. A. P. Turner, M. Am. Soc. C. E. (by letter).—The writer agrees with the criticism of this report made by the minority. He will go further, and state that, in his judgment as a concrete constructor, the report, contrary to Mr. Worcester's assertions, contains dangerous theory which, as interpreted by the practical constructor, will lead to disastrous results. Mr. Turner.

Before taking up this point, the make-up of the Committee will be reviewed. The earlier part of the report shows that none of the members of the Committee is interested commercially in reinforced concrete. To the layman this leads at once to the question: Are the members of the Committee familiar enough with the subject of reinforced concrete to pass intelligently on it?

The idea presented, that because the members were not interested commercially, the reports would be more accurate and unbiased than if made by those who are more familiar with, and commercially engaged in, this work, is somewhat amusing when one considers the fact that those commercially engaged in the actual construction of reinforced concrete frequently are forced to guarantee their work to the extent of every dollar they possess; that men who are putting up such guaranties are far too intelligent to suppose for a moment that they can humbug the law of gravitation, as regards the strength of work, which must be tested by placing a specified test load upon it. If men of this class, commercially engaged in this line of work, should allow their judgment to be influenced by anything short of well-determined facts, it is certain that they would soon disappear from the commercial field, through bankruptcy.

Mr. Turner.

Mr. Worcester calls attention to the long time which has been spent by this Committee in preparing this so-called progress report, and those interested in the industry naturally infer that this may be accounted for on the ground that those who were delegated to serve on this Committee, as stated in the report, were not commercially interested and hence were forced to commence their investigations at the crude primitive or elementary stage of this class of work. Looking into this report carefully, it seems to the writer that the members have not even progressed thus far in their investigations; that they have failed to recognize that they were dealing with a new material, one which is used advantageously in monolithic form; that when used in this way it can be advantageously strained in many directions; that when so strained the laws of bending, as compared with simple beams, are radically different; that the relative efficiency of the two distinctive materials—steel and concrete—making up the combination, have to be considered; that the experiments conducted under the direction of this Committee did nothing whatever toward throwing light on the relative efficiency of the two materials when strained in any direction except parallel to the bars; that, in reference to slabs reinforced in two directions, a pretense of presenting an elastic theory has been made, with no idea of checking the elastic theory by the elastic deportment of the material.

Had this been done, the Committee would not have presented a formula for multiple-way reinforcement which requires three times the steel necessary to develop fully the rated capacity. Such a formula is dangerous in practice, because when the practical constructor, using it and understanding its derivation, finds that the test capacity of the slab supported on four sides and reinforced in two directions, is three times as great as it could be calculated by the formula given by this Committee, he assumes that the slab reinforced in one direction may also be tested to three times its calculated capacity. Accordingly, he tries it, and a failure results, due to the ignorance displayed by this Committee in publishing a formula so grossly in error.

The writer is astonished indeed that this Special Committee, which claims that its members have no commercial interest in the reinforced concrete industry, should have any expectation whatever that its report would be considered authoritative, by those engaged in the business, without presenting test data to uphold its opinions. This point is brought out strongly in the Minority Report.

The members presenting the Minority Report are structural steel engineers; men who, as a rule, do not, in their line of business, accept anyone's opinion on the strength of materials unless that opinion is upheld by adequate test data, and they have applied the same line of reasoning to this report.

The writer considers the testing done at St. Louis as of absolutely

no value to the Profession at large, due to the fact that it was not conducted along lines which would throw any light on the strength of actual construction, save and except construction which is made up in independent units, a method of putting up work, which, while it may be practical for light shed construction, is entirely out of place in any structure of magnitude or in any structure carrying heavy loads. Mr. Turner.

If the members of the Society wish to retain the respect of the practical concrete constructor and contractor on the subject of concrete and reinforced concrete, they must bear clearly in mind the fact that men of this latter class, while they may not do much figuring, are likely to get very decided opinions regarding the strength of the construction by the simple method of the Swede foreman, who, when asked what a certain joist would carry on a 12-ft. span, stated that he did not know how much it would carry, but that it would carry 1 600 lb. He was asked, "how do you know"? He replied: "I tried it; at noon hour, I set it up and had eleven men stand on it."

So with the concrete constructor. He places a certain load on his structure, and he knows that, with the reinforcement used, it will carry that load, and no amount of figuring which shows the construction to be only one-third as strong, is likely to be considered favorably by him.

If the members of the Committee were to take into consideration this method of reasoning, they would be more careful in presenting formulas so grossly in error.

The writer does not wish the foregoing criticism to be misunderstood as in any wise underrating the desirability, and, in fact, the necessity for accurate methods of design. However, in his practice, he has found that only by the experimental determination of practical constants can the strength and also the elastic behavior of the construction under load be readily determined, and that this cannot be accomplished by the determination of such elastic constants as have been determined in experiments commonly conducted heretofore on small specimens.

The writer has covered this matter somewhat more fully in a recent publication on concrete steel construction, and believes it unnecessary to go over the same ground a second time in this discussion.

MEMOIRS OF DECEASED MEMBERS

HENRY WALLER BRINCKERHOFF, M. Am. Soc. C. E.*

DIED SEPTEMBER 7TH, 1909.

Henry Waller Brinckerhoff was born on May 22d, 1845, in Sing Sing, now Ossining, N. Y., and was the eldest son of Isaac Brinckerhoff, Medical Director, United States Navy, and Mary Gordon (Waller) Brinckerhoff.

His ancestry, on his father's side, was Dutch, and on his mother's, English and Scotch. The Brinckerhoffs were among the earliest settlers of the United States, coming from Holland in 1638, and many descendants of the name in New York and neighboring States represent the vigor of this stock, members of which took part in the Revolutionary and subsequent wars.

The Scottish Gordons have been conspicuous in history, and one of that name, General James Gordon, Mr. Brinckerhoff's great-grandfather, was present at the Battle of Bennington. Mr. Brinckerhoff's father served in the Civil War, and he himself, while yet a student, served three months with the 12th Unattached Company of Massachusetts Volunteer Militia.

Mr. Brinckerhoff's early years were spent in Cambridge, Mass., where he obtained his education at the Latin School, Harvard College, and the Lawrence Scientific School of Harvard University. His first work was as an Assistant in the Civil Engineer's office at the United States Navy Yard, Brooklyn, N. Y. In 1869, he was appointed a Civil Engineer in the Navy, and was stationed at the same yard, where he served for some time, constructing quay walls and other naval work. Upon leaving that post, on January 1st, 1870, he was appointed Principal Assistant Engineer on the Jersey City Water-Works, under the late Mr. John F. Ward, in which capacity he designed the pumping machinery and made the plans for the high-service and new reservoir, also supervising sewer construction and other municipal work.

With the late Mr. Samuel E. McElroy, Mr. Brinckerhoff was engaged upon the Kings County town survey, and afterward made a hydrographic survey of the Passaic River, from Newark to Rutherford Park, for the late General John Newton, Chief of Engineers, U. S. Army, Hon. M. Am. Soc. C. E. In 1874 he was engaged with Messrs. Davidson and Mars, Contractors for the heating arrangements and elevators in the New York City Post-Office Building, his work being the design and erection of the telescopic elevators for that building. Mr. Brinckerhoff remained with this firm for some years, and was

* Memoir prepared by Charles H. Myers, M. Am. Soc. C. E.

afterward engaged for a short time in steam-heating work for Mr. Wyllys H. Warner.

In the autumn of 1877 he took charge of the construction of the foundations of the Gilbert Elevated Railroad from Grand Street to Morris Street, New York City, under the Chief Engineer, the late Mr. William F. Shunk. He performed other work in connection with that structure, and in the following year he inspected the shop work for the Ninth Avenue "Joint Structure" Elevated Railroad. In June, 1879, under the direction of Major George W. McNulty, M. Am. Soc. C. E., Mr. Brinckerhoff designed and inspected the construction of the iron street bridges in the Brooklyn approach of the East River Bridge—the first bridge across the East River, now officially known as "The Brooklyn Bridge."

In 1881, Mr. Brinckerhoff resigned his position on this work, to take charge of the building work of the New York Steam Company, under the late Charles E. Emery, M. Am. Soc. C. E., Engineer and Superintendent for said company. He designed and superintended the erection of the large boiler station on Greenwich Street, New York City, and, after this work was finished, he assisted the late James B. Eads, M. Am. Soc. C. E., in his plans for the proposed ship railway across the Isthmus of Tehuantepec, Mexico. For some time afterward, Mr. Brinckerhoff was Managing Editor of *The Engineering Record*, and, in 1889, made the earliest report on the failure of the South Fork Dam which caused the destruction of Johnstown, Pa.

When the plans for the Broadway Cable Railroad were to be made, Mr. Brinckerhoff was called by Major McNulty, Chief Engineer, to take charge of the office work, and he remained in that position until the road was completed and put into operation. He then began his practice as a Consulting Engineer, being thus engaged until illness compelled a cessation of his active labors shortly prior to his end. His death was caused by an arterial affection which assumed a serious phase in the last few months of his life.

A characteristic of Mr. Brinckerhoff's work was a thoroughness which was noticeable in all his undertakings. As Consulting Engineer, his advice was sought in regard to the foundations of the Johnston Building, at Broad Street and Exchange Place, and other heavy buildings in New York City, besides various other works in the vicinity. Mr. Brinckerhoff had professional experience in both civil and mechanical engineering.

He was most patient and considerate in his dealings with all who came in contact with him, while his honesty and candidness made him outspoken whenever occasion required. His, too, was a kindly and sympathetic nature, with deeply religious sentiment. For a long time he was a Deacon of the Central Congregational Church in Brooklyn, N. Y., and he took much interest in all missionary work. To the

stranger from distant foreign countries, he gave a welcome, often with timely assistance. His church will feel his loss greatly.

Mr. Brinckerhoff was President of the Bedford Dispensary and Hospital, of Brooklyn, N. Y.; Vice-President of the Congregational Club, of Brooklyn; a Member of the Holland Society, and of the Masonic Order. He married, at Cambridge, Mass., on December 25th, 1866, Edith Adelaide Barry, who died on April 19th, 1897. Her loss, and that of his eldest daughter, the wife of the Reverend Winthrop B. Greene, three years later, were heavy blows, which left their lasting effect upon him. A son, Henry Gordon Brinckerhoff, and a daughter, Mrs. Herbert G. Hanford, survive him.

Mr. Brinckerhoff was a loyal friend and a genial companion, and those who asked his advice received cheerful hearing and sincere counsel. Naturally, he took great interest in the early history of his native State, especially in the period of the Dutch occupation. He will be remembered among his acquaintances for the possession of a fund of humor and store of varied information. In his closing days, his patience and serenity never forsook him, and his passing away was peaceful and painless.

Mr. Brinckerhoff was elected a Member of the American Society of Civil Engineers, on November 7th, 1883.

ELSTNER FISHER, M. Am Soc. C. E.*

DIED OCTOBER 12TH, 1909.

Elstner Fisher, the son of Mr. and Mrs. William Mack Fisher, was born in Cincinnati, Ohio, on October 9th, 1853. He received his education at private schools in Cincinnati, and at the Central High School of Philadelphia (afterward the University of Pennsylvania), from which he was graduated with the degree of Bachelor of Arts.

In 1872, Mr. Fisher was appointed to the United States Naval Academy, from which he was graduated in 1876. He was assigned to the U. S. S. *Vandalia,* as Midshipman, and for two years served on the staff of General Grant, who made that vessel his headquarters during the Mediterranean part of his trip around the world. Mr. Fisher also made a four years' cruise on the *Wachusetts,* around the Horn and up the Pacific Coast as far as San Francisco.

In 1882, he was detailed as Assistant to the U. S. Coast and Geodetic Survey, and, in that capacity, made deep-sea soundings from Boston, Mass., to Rio de Janeiro, Brazil. Owing to the illness of the Commanding Officer, Mr. Fisher had charge of the work of the expedition. He was afterward engaged in making surveys of Delaware River and Bay, Cape Romaine, South Carolina, and Long Island Sound.

In January, 1884, Mr. Fisher resigned his commission in the Navy and entered the employ of the Pennsylvania Railroad, serving as Chainman, Rodman, and Transitman on the location of the Schuylkill Valley Railroad. On the completion of this work, he was transferred to Altoona, Pa., as Assistant Engineer in the Maintenance-of-Way Department, being engaged on the location and construction of various branch lines and in general engineering work. In July, 1886, Mr. Fisher was appointed Assistant Supervisor on the Monongahela Division of the Pennsylvania Railroad, from Pittsburg to Monongahela City.

In 1887 he accepted the position of Assistant Engineer on the Michigan Central Railroad, with headquarters at Detroit, Mich. He was afterward transferred to the Operating Department and served in various capacities, namely, Night-Yardmaster, Yardmaster, Assistant Trainmaster, Trainmaster, and Assistant Superintendent of the Middle Division, with headquarters at Jackson, Mich.

In November, 1897, Mr. Fisher was appointed General Superintendent and Chief Engineer of the Toronto, Hamilton and Buffalo Railway, which position he resigned shortly before his death on October 12th, 1909.

* Memoir prepared by R. L. Latham, Esq., Chief Engineer of the Toronto, Hamilton & Buffalo Ry. Co.

In 1883, he married Miss Sarah Burt, the daughter of Mr. and Mrs. Wells Burt, of Detroit, Mich.

Mr. Fisher was elected a Junior of the American Society of Civil Engineers on April 3d, 1889, an Associate Member on June 2d, 1897, and a Member on March 5th, 1902.

He was also a Member of the Canadian Society of Civil Engineers; the Transportation Club of New York; the Buffalo Club of Buffalo; the Hamilton Club of Hamilton; the Brantford Club of Brantford; the Royal Hamilton Yacht Club; the Caledon Mountain Trout Club; and the Barton Lodge, A. F. and A. M., and Hiram Chapter, R. A. M.

WILLIAM CHARLES KERNOT, M. Am. Soc. C. E.*

DIED MARCH 14TH, 1909.

William Charles Kernot was born at Rochford, Essex, England, on June 16th, 1845. With his father, he went to Australia in 1851—the year in which gold was discovered in large quantities in Victoria.

He received his primary education at the Geelong National Grammar School and, in 1861, matriculated at Melbourne University, where he passed with distinction through the Arts and, subsequently, the Engineering Courses, his name appearing seven times on the honors list, three exhibitions and one scholarship being also awarded to him.

Professor Kernot began his engineering practice on the Geelong Water-Works, and this was followed by work on the Coliban Water-Works and in the Railways and Mines Departments.

In 1869, he began his connection—in the capacity of lecturer—with the staff of Melbourne University. In 1883 he was appointed Professor of Engineering, a position which he filled, and with which he was identified, until his death. In 1888 he also fulfilled the functions appertaining to the Chair of Natural Philosophy.

Professor Kernot served six times as President of the Victorian Institute of Engineers, in 1886, 1890, 1897, 1898, 1906, and 1907. He was also a Member of the Institution of Civil Engineers, London, and of the Victorian Institute of Surveyors. He was a Vice-President of the Royal Geographical Society of Australasia, a Fellow of the Royal Geographical Society of England, for several years President of the Royal Society of Victoria, and a Past-President of the Engineering Section of the Australasian Association for the Advancement of Science.

Among the numerous public and advisatory positions filled by Professor Kernot at various times may be cited the following: Member of the Municipal Surveyors Board for forty years—serving thirty years as Chairman of that board; Member of the old Board of Public Health; Member of the 1880-1881 Melbourne Exhibition Committee; and Chairman of Juries for the Melbourne Jubilee Exhibition, 1888. He was a Member of Council and one of the Trustees of the Melbourne Working Men's College, of which school he was also President from 1889 to 1899. He was also a Member of the following Boards: Victorian Railway Enquiries and Commissions; New South Wales Bridge Commission; Royal Commission on the Stability of Victorian Railway Bridges; Board of Visitors to the Melbourne Observatory; Yarra (River) Floods Board; Telegraph and Telephone Tunnel Board (1886); Melbourne Drainage Commission (1889); Board of

* Memoir prepared by E. Manchester, Assoc. M. Am. Soc. C. E.

Examiners for Water Supply Engineers; Directorate of the first Australian Electric Light Company; Trustee of the Royal Park, and numerous other offices. Professor Kernot made three trips to Europe and America, and was in South Africa during the progress of the Boer trouble.

He made many donations to public purposes, among others being liberal benefactions to the Working Men's College, also the sum of £1 000 to found a Scholarship in Metallurgy, and a sum of £2 200 to establish a Scholarship in Chemistry and Natural Philosophy, at Melbourne University.

Without doubt Professor Kernot was one of the kindliest and most genial of men, at all times absolutely unsparing of himself in the interests of others—one of Nature's own noblemen—and his death has caused a gap in the life of Victoria, which will be hard to fill.

In public life it would be difficult to say with what engineering function, with what important engineering movement in Victoria for more than forty years, he was not identified, but his chief work—his life's work—was associated with his University duties. His students received more than technical training, for Professor Kernot's genial kindly nature invited confidence, and his advice left a lifelong impression upon them. Above all he was so absolutely an upright man, that every day the example he set of what an honorable member of an honorable profession should strive to attain was above price to his students.

Professor Kernot was unmarried. He was elected a Member of the American Society of Civil Engineers on March 6th, 1889.

HENRY BROWN RICHARDSON, M. Am. Soc. C. E.*

DIED AUGUST 21ST, 1909.

Henry Brown Richardson was born on August 23d, 1837, in Winthrop, Me. He was the son of the Reverend Henry Richardson and Eunice Farley Thurston. Among his ancestors and relations were many historic New England names. An adventurous and healthy boyhood, passed along the valley of the Androscoggin River and among the foothills of the White Mountains, laid the foundation for his understanding of Nature's operations, and gave him the bodily strength and endurance which subsequently formed part of his professional equipment.

Having acquired the education which the local schools extended, he left home for broader fields of work and life than the neighborhood afforded. His tastes and acquirements led him to seek employment of a technical and constructive character, which he first found in engineering offices in Portland, Me., and afterward, as his wanderings extended, in Providence, Boston, and in Milwaukee, Chicago and other Western cities. In the intervals in such employment, he taught school in Illinois and elsewhere.

In 1860 Mr. Richardson reached Louisiana, where he was destined to find his future home and life work. He readily secured employment with the railroads and other industrial developments in that region, and particularly under the State authorities then in charge of the levee system of the Mississippi River. These engagements were soon interrupted by the outbreak of the Civil War.

In April, 1861, Mr. Richardson enlisted in an infantry company which was promptly ordered to Virginia. He was then immediately detailed on engineering duty, and served successively on the staffs of Generals Dick Taylor, Early, and Ewell. His services, particularly in rapid, daring, and accurate reconnaissance, were distinguished, and led to his steady promotion until, by the time the war was half over, he had attained the rank of Major of Engineers. He was desperately wounded at Sharpsburg and Gettysburg, and, from the latter field, was carried to Johnson's Island where he remained a prisoner until almost the close of the war.

Immediately on the restoration of peace Mr. Richardson returned to Louisiana and resumed his engineering practice, of which the reclamation of the Mississippi Valley from overflow formed a large and important part, and, becoming his life work, claimed more and more of his service.

In 1877, on the reorganization of the State levee system, he was appointed, with B. M. Harrod, Past-President, Am. Soc. C. E., and the

* Memoir prepared by B. M. Harrod, Past-President, Am. Soc. C. E.

late T. S. Hardee, M. Am. Soc. C. E., to form a Board of Engineers having control of the levees of Louisiana. In 1881 he was made Chief Engineer of this Board, which position he retained for twenty-three years, until 1904. In this office he was charged with the improvement and maintenance of 1 500 miles of levee containing 150 000 000 cu. yd. of embankment, and protecting 15 000 sq. miles of territory from overflow, at an average annual expenditure of more than $1 000 000.

Mr. Richardson's duties as State Engineer brought him in constant and close relations with the conflicting interests of the work, the riparian owners, and the numerous contractors, but were discharged without friction and in a manner which inspired all concerned with the most absolute confidence in his knowledge, judgment, and fairness. The evidence of the wisdom of his administration and the confidence with which he inspired all with whom he dealt is that he retained an office which was subject to change at every change of State administration for twenty-four years, until he resigned it to accept service under the United States as a Member of the Mississippi River Commission.

His mental vigor and readiness, great physical endurance and resolution, equipped him well for a branch of professional work which is familiar with dangers, exposure, and emergencies, and where for months great interests hang in jeopardy from floods. His strong and active powers of observation, trained by years of experience, gave him great authority as a student of alluvial streams, and enabled him to render probably the most eminent service of any of his colleagues in the great work of reclaiming the Mississippi Valley from overflow.

In 1904 Mr. Richardson resigned as Chief Engineer of Louisiana to accept membership on the Mississippi River Commission, for which it was universally conceded that his special experience eminently fitted him.

With Majors Crittenden and Dabney, he was a member of the Board to report on the "Rectification of the Sacramento and San Joaquin Rivers and their principal tributaries." He was also on the Board for the Drainage of New Orleans, and was in conference on many other engineering projects of magnitude.

While this memoir is chiefly concerned with Mr. Richardson's career as a Civil Engineer, it would give but a meager idea of his character and influence if no mention were made of the personal traits for which he was beloved by all who had the privilege of his intimacy, which earned him the trust and respect of the community.

Mr. Richardson's character was one of peculiar modesty and reserve, combined with unusual firmness and courage. He was patient and unselfish to a degree. He was incapable of engaging in any struggle for personal distinction or advantage, and shrank from any prominence which was not based on the merit of his work. He asked

no favor, and all his work and fame came to him as the result of successful achievement and established character.

In 1867, Mr. Richardson married Miss Anna Howard Farrar, of Tensas Parish, La., and the union proved to be one of great happiness. There were born to them nine children, of whom all but the eldest survive. Of six boys, one is an Associate Member of this Society, and three others are in engineering employment.

Mr. Richardson was elected a Member of the American Society of Civil Engineers on May 7th, 1879, and served as a Member of the Board of Direction from 1900 to 1902.

ALBERT STANLEY RIFFLE, M. Am. Soc. C. E.*

DIED OCTOBER 28TH, 1909.

Albert Stanley Riffle was of German-Swiss parentage. He was born on March 26th, 1860, at Somerset, Pa., and died at Sierra Madre, Cal., on October 28th, 1909. He was the son of William and Mary E. (Speicher) Riffle.

At the age of 11 years, he removed with his parents to a farm near Springhill, Kans. Here his early life was spent until his entrance as a student into the Kansas State University, from which institution he was graduated with the degree of B. S., in 1884. From this time until 1902 he was actively engaged in the practice of the profession of civil engineering. At the latter date commenced his long and valiant contest with the insidious ailment which finally caused his death.

Mr. Riffle's principal professional engagements were as follows:

From 1884 until 1889 he was in the employ of the Northern Pacific Railroad on the construction of the Cascade Division, most of the time under Virgil G. Bogue, M. Am. Soc. C. E., who was then acting as Principal Assistant Engineer on that road. Mr. Riffle served in various grades, both in active charge of construction work and on location, most of his services, however, being rendered as Superintendent of Bridges and Buildings. In this capacity he prepared the designs of practically all the truss and trestle bridgework, and carried to completion the construction of the bridge across the Columbia River at Pasco. He was placed in charge of this latter work after the substructures and some falsework were in position, succeeding the late L. L. Buck, M. Am. Soc. C. E., who initiated the work.

In 1889-90, Mr. Riffle acted as Principal Assistant Engineer to his brother, Franklin Riffle, M. Am. Soc. C. E., then Chief Engineer of the Oregon and Washington Territory Railroad, and, in this capacity, had charge of the planning and erection of most of the bridgework on that road.

The years 1890-92 were spent in Peru, whither he was sent at the suggestion of Mr. L. L. Buck, by William R. Grace and Sons, to take charge of the erection of the great Verrugas Viaduct, built to replace one erected at an earlier date by Mr. Buck, which had been destroyed by a great flood. This work, which was quite fully described in the engineering journals at the time, was successfully carried through under difficult and very unusual conditions arising from the mixed and uncertain character of the available labor, and, chiefly, because of the great fatality from Verrugas fever among all engaged on the work.

* Memoir prepared by Arthur L. Adams, M. Am. Soc. C. E.

The writer recalls that of four skilled erecting foremen taken by Mr. Riffle from New York for this work, only one returned; and that of three clerks in his office on the work, he buried two. Mr. Riffle himself suffered two attacks of the fever and also contracted the Panama fever on his return, and continued to suffer from the effects of this experience for years afterward.

From 1892 to 1895 he was associated with his brother and others under the name of the Oregon Bridge Company, with offices at Portland, Ore. This company was engaged in general contract work, chiefly in the construction of highway bridges throughout the States of Oregon and Washington.

In the years 1897 to 1899, Mr. Riffle was in the employ of the Valley Road, now constituting the Santa Fé's System in California, north of Bakersfield, being, for a part of this time, in charge of the design of all structures, under W. B. Storey, Jr., M. Am. Soc. C. E., and later in charge of the Franklin Tunnel and other important structures, under W. C. Edes, M. Am. Soc. C. E., the Assistant Chief Engineer.

From 1899 to the time when ill health compelled his discontinuance of active work, Mr. Riffle was in the employ of the Excelsior Wooden Pipe Company, of San Francisco, and was engaged in the construction, as Contractor, of wooden stave pipe and other water-works structures. During this period he had charge, under D. C. Henny, M. Am. Soc. C. E., the General Manager, of carrying out numerous contracts on important pipe-line and reservoir construction. During his active career Mr. Riffle filled many other important engagements, such as experting the designs for a number of the World's Fair Buildings for the Columbian Exposition at Chicago, under the direction of Mr. Bogue, and the making of expert valuations on water-works properties at Oakland, Cal., and at Albuquerque, N. Mex.

As an engineer Mr. Riffle possessed attainments of a very high order, much higher indeed than the scope of his practice ever gave him opportunity to exemplify. These attainments, because of some natural diffidence, were best appreciated by the men of his own profession, those standing high in public and professional esteem often availing themselves of his special knowledge. He combined, in an unusual degree, thoroughness and knowledge of detail, with an exceptional capacity for rapid and successful execution of work.

His contribution to professional progress is represented chiefly by a paper, prepared by Mr. Franklin Riffle and himself, descriptive of the successful laying of a large submerged pipe across the Willamette River at Portland, Ore.* His greatest contribution, however, is undoubtedly to be found in the power of his example of faithful service and successful accomplishment in the discharge of the responsibilities

* *Transactions*, Am. Soc. C. E., Volume XXXIII, p. 257, 1895.

which came to him in the practice of his profession during the years of his active life.

Personally, Mr. Riffle was a man of sterling character, and possessed of qualities of heart and mind which greatly endeared him to his friends, and indeed to all who were favored with his intimate acquaintance. His splendid pluck and sustained buoyancy of spirit during the long years of unavailing struggle against disease, prior to his death, gave touching evidence of the inherent strength of the man, and of his faith in the overruling Power. He is survived by his wife, Mrs. Bella Love Riffle.

Mr. Riffle was elected a Member of the American Society of Civil Engineers, on May 7th, 1890.

HORATIO SEYMOUR, Jr., M. Am. Soc. C. E.*

DIED FEBRUARY 21ST, 1907.

Horatio Seymour, Jr., was born in Utica, N. Y., on January 8th, 1844. He was the son of John F. and Frances A. (Tappan) Seymour, and the nephew of Horatio Seymour, a former Governor of the State. He was educated in the Academy at Utica, and at Yale College, from which he was graduated in 1867. In 1869-70 he attended Yale Scientific School.

His first engineering service was as an Assistant on the Cazenovia and Canastota Railroad. Afterward, he was with the Seneca Falls and Sodus Bay Railroad, the Wellsville and Lawrenceville Railroad, and was Chief Engineer of the Cowanesque Valley Railroad. In 1873 and 1874, he was with the Fall Brook Coal Company and the Buffalo Coal Company. From October 1st, 1874, to December 31st, 1875, he was an Assistant Engineer in the State works of New York on the Erie Canal, and was afterward engaged on the topographical survey of the State. He became State Engineer and Surveyor of the State of New York, on January 1st, 1878, when he was 34 years old, and served two terms of two years. He was selected for that office as one of the representatives of the spirit of vigorous reorganization of the management of the great public works of that State, which had been inaugurated three years before by a Governor who notably elevated the standard of public service, and whose work in this reform had been ably enforced by another Member of this Society, the immediate predecessor of Mr. Seymour.

Mr. Seymour then went to Marquette, Mich., where he engaged in general practice, but soon took charge of the interests of the Michigan Iron and Land Company, which owned large land properties, including iron mines and extensive tracts of fine timber. The management of these properties occupied him fully until 1903, when he returned to Utica, where he resided until his death.

Mr. Seymour's notable professional service was in connection with the State works of New York. While an Assistant Engineer, his declination to certify certain estimates without examination, and his protests against the unlawful demands of contractors, were, in part, the basis of an extended investigation of canal frauds and a radical reform in methods.

As State Engineer, Mr. Seymour conducted the department with ability and economy. His four reports are notable State papers. The first, written after a year's incumbency of the office, discusses the value of canals and water routes for internal commerce, their relation to

* Memoir prepared by John Bogart, M. Am. Soc. C. E., and Alfred Noble, Past-President, Am. Soc. C. E.

railroads, and their influence in maintaining reasonably low transportation rates. It calls attention to the danger that traffic might be diverted to the water route through the Valley of the St. Lawrence, where the canals are being enlarged and deepened, and argues that a method must be found to lessen the cost of carrying freight by the Erie Canal route, and thus save the commerce to the State and City of New York. This could be done, either by increasing the tonnage of the boats or by increasing their speed. The report advises that the depth of water in the canal should be increased 1 ft. This would add to the speed of the boats and enable them to make an additional trip during the navigation season, or it would add largely to their carrying capacity. The use of power to operate the locks is advocated as an additional means for saving time in transit. The report shows that the Erie Canal had paid into the Treasury of the State about $42 000 000 more than its cost, including all expenditures for repairs and management.

In his second yearly report, Mr. Seymour repeats and enforces the recommendations of the first. In his third report he shows that the market of New York City gives value to property throughout the State, that that city pays about one-half the taxes of the State, and that commerce must be saved to the city and the State. The canals furnish a cheaper means of transportation than any other method. By making the cost of this still lower, commerce will be saved. The plan of adding 1 ft. to the depth of water commends itself to the boatmen, because it requires no outlay on their part, and this increase in depth can be secured by the expenditure of about $1 000 000. Mr. Seymour's fourth and last report advocates the abolition of tolls on the State canals.

The plan of adding to the capacity of the canals by an increase of 1 ft. in the depth of water was seriously considered during subsequent years, and was always known as the Seymour plan. It was not carried out, but the information gathered by Mr. Seymour, and the suggestions made in his reports, were, in a large way, elements in the extended studies which finally resulted in the adoption of the barge canal project, now in active construction.

Mr. Seymour's service as State Engineer was marked by professional ability and excellent business management. These qualities characterized also his management of the large interests subsequently under his care in Michigan.

In 1880 Mr. Seymour married Miss Abigail Adams Johnson, who, with a son, Horatio, and a daughter, Mrs. Henry St. Arnauld, survives him. He was a member of the Sons of the Revolution and of the Society of the Cincinnati.

Mr. Seymour was elected an Associate of the American Society of Civil Engineers on January 8th, 1873, and a Member on April 7th, 1880.

CARL WILLIAM BIRCH-NORD, Assoc. M. Am. Soc. C. E.*

DIED SEPTEMBER 15TH, 1909.

Carl William Birch-Nord was born in Christiania, Norway, on October 7th, 1880. He was graduated from the Christiania Elementary Technical School in 1899, and afterward completed a post-graduate course in Electrical Engineering at the Institute of Technology at Bingen, Germany.

In 1902 Mr. Birch-Nord came to the United States and settled in Chicago, Ill., where he held positions as Draftsman with Burnham and Company, the Metropolitan Elevated Railway Company, etc. In June, 1903, he entered the service of the American Bridge Company, as Estimator and, later, as Assistant Engineer, in the Estimating Department of the Chicago Plant, where he remained until his death on September 15th, 1909.

On May 15th, 1907, Mr. Birch-Nord was married to Miss Marie Halvorsen, who survives him.

Mr. Birch-Nord was elected a Junior of the American Society of Civil Engineers on October 30th, 1906, and an Associate Member on November 4th, 1908. He recently contributed a paper on "The Design of Elevated Tanks and Stand-Pipes" to the *Transactions* of the Society. He was also a Member of the Western Society of Engineers, and of the Technical League.

* Memoir prepared by Albert Reichmann, M. Am. Soc. C. E., and J. H. Hoff, Esq.

ERNEST STEARNS BALL, Jun. Am. Soc. C. E.*

DIED NOVEMBER 22D, 1909.

Ernest Stearns Ball, the son of Henry Eugene and Julia (Blandin) Ball, was born in Gardner, Mass., on August 17th, 1882. He prepared for college at the Gardner High School, and in 1899, entered Norwich University from which he was graduated in 1903 with the degree of B. S. in Civil Engineering.

From June, 1903, to June, 1904, Mr. Ball was employed as Draftsman by the Riter Conley Manufacturing Company, of Pittsburg, Pa., and from June, 1904, to July, 1905, he was with the firm of Post and McCord, of New York City, on steel construction work.

From that time until December, 1906, Mr. Ball was in business as a Contracting Engineer, designing and constructing power-houses, tanks, foundations, etc. He was afterward engaged in miscellaneous work in Northfield and Roxbury, Vt., until May, 1909, when he was appointed Assistant Engineer in the Maintenance-of-Way Department of the Missouri Pacific Railroad, which position he held until his death, in the Missouri Pacific Railroad Hospital, at St. Louis, Mo., on November 22d, 1909.

Mr. Ball is survived by his wife, Myrta M. (Tarbell) Ball, and one son, Theodore Macklin Ball.

Mr. Ball was elected a Junior of the American Society of Civil Engineers on May 2d, 1905.

* Memoir prepared by Arthur E. Winslow, Assoc. M. Am. Soc. C. E.

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